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Dam Maintenance and Rehabilitation II

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Foreword

In October of 2002, SEPREM (Sociedad Española de Presas y Embalses, or Spanish Society of Dams and Reservoirs), together with the Dirección General del Agua del Ministerio de Medio Ambiente (Water Directorate of the Ministry of the Environment), organised in Madrid (Spain) the First International Congress on Dam Maintenance and Rehabilitation. The bulleting which announced the Congress stated the following:

"Dams, like any other man-made construction immersed in Nature, are originally conceived to stand against a number of actions. As dams age, they are subject to a series of external agents and processes which tend to deteriorate the qualities with which they were originally conceived to stand against these actions. Deterioration processes may affect the foundation on which they rest, the materials with which they were built, or result in damage of the outlet works. Reservoir sedimentation or water quality issues within the reservoir also give raise to problems that must be addressed. In addition to the previous factors, it will be necessary in many cases to respond to increased safety standards, either in the structural or hydrological fields.

Criteria to establish how dams and reservoirs deteriorate and the magnitude of this deterioration vary from one country to another, but it is clear anyhow that works of maintenance, rehabilitation, retrofit and decommissioning constitute more and more an important field of activity in every country which has in place a system of dams of certain age, among which Spain is a clear example. Dam Owners will necessarily have to increase their budget allocation in order to reach the increasing operation, supply and security measures which are being established. A substantial number of scientists and engineers, as well as consulting and construction companies will be involved in these activities".

The time passed since the Congress, far from making these words less valid, has only made them more evident and pressing, making some related concepts more obvious such as the incidence of climatic change in the management of water resources, and the need for sustainability.

In view of all this, SEPREM, again in collaboration with the Dirección General del Agua del Ministerio de Medio Ambiente y Medio Rural y Marino organizes the Second International Congress on Dam Maintenance and Rehabilitation, to take place in Zaragoza (Spain) during November 2010. This Congress serves as a forum, not only to Exchange knowledge and debate experiences of the diverse and complex field of dam maintenance and rehabilitation, but also to share the techniques and conservation activities which may help to prevent or delay the need for major capital maintenance. The Congress will also serve to convey the experience of the Spanish practitioners and companies to the international community of professionals in this field.

According to the Organizers and Editor's point of view, the contributions to the Congress here presented represent the highest level of development achieved in techniques concerning dam retrofitting and conservation. A cursory look at the Table of Contents shows how diversified the topics considered are. Both Editors and Organizers hope that the present Proceedings will become a helpful device in the achievement of present and forthcoming safety standards in dams by means of a proper maintenance, repair and rehabilitation.

Zaragoza, November 2010

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Prólogo

En octubre de 2002 la Sociedad Española de Presas y Embalses (SEPREM), organizó conjuntamente con la Dirección General del Agua del entonces Ministerio de Medio Ambiente el 1er Congreso Internacional sobre Conservación y Rehabilitación de Presas que tuvo lugar en Madrid (España). En su convocatoria inicial se expresaba:

"Las presas, como todas las obras humanas inmersas en la naturaleza, se hallan sometidas desde su construcción a una serie de agentes que tienden a mermar las cualidades iniciales con que fueron concebidas para hacer frente a unas determinadas acciones. Dichos agentes pueden afectar al cimiento donde apoya la presa, a los materiales con que está constituida o incidir en el deterioro de los órganos de desagüe. La colmatación del vaso o la contaminación de las aguas o del mismo vaso pueden dar lugar a problemas que también requieren actuaciones. En otras ocasiones, y en paralelo con lo antes señalado, será necesario dar respuesta a unos niveles de seguridad más estrictos, bien de tipo estructural, bien de carácter hidrológico.

Los criterios para establecer cómo los embalses y las presas sufren deterioros y en qué magnitud varían evidentemente de unos países a otros, pero en cualquier caso todo parece indicar que las labores de conservación, rehabilitación, adecuación y puesta fuera de servicio de presas y embalses van a constituir un campo de actividad muy importante en todos los países donde el parque de presas tiene una cierta antigüedad, entre los que España ocupa un lugar destacado. Necesariamente los Titulares de presas tendrán que ir incrementando sus partidas presupuestarias para poner a punto su patrimonio hidráulico y un buen número de científicos, técnicos y empresas consultoras y constructoras se verán involucradas en estas actividades."

El tiempo transcurrido desde entonces no sólo no ha hecho perder validez al planteamiento anterior sino que, por el contrario, ha potenciado su vigencia y actualidad, haciéndose más patentes conceptos relevantes que inciden en la cuestión, como son, por ejemplo, la incidencia del cambio climático en la gestión de los recursos y la búsqueda de la sostenibilidad de las actuaciones.

Por todo ello, la Sociedad Española de Presas y Embalses (SEPREM), junto con la Dirección General del Agua del Ministerio de Medio Ambiente y Medio Rural y Marino convoca el 2º Congreso Internacional sobre Conservación y Rehabilitación de Presas, a celebrar en Zaragoza (España) durante el mes de noviembre de 2010. Este Congreso sirve de foro donde transmitir y debatir, no sólo los conocimientos y experiencias de las diversas y complejas facetas de la conservación y rehabilitación de presas y embalses, sino también aquellas técnicas y actuaciones preventivas y de buena práctica tendentes a evitar o demorar la ejecución de actuaciones importantes. Al tiempo, quiere transmitir a la comunidad internacional dedicada a estas actividades las experiencias y capacidades de los técnicos y de la industria española en este campo.

Las comunicaciones recibidas en el marco de este Congreso y que en esta publicación se recogen, constituyen, a juicio de la Organización y de los Editores, una revisión actualizada del estado del arte en el que, a escala internacional, se encuentra la técnica y práctica en el campo de la conservación y mantenimiento de presas y confían en que puedan contribuir a la divulgación del conocimiento sobre este área y, de alguna manera, ayudar a que las presas existentes puedan seguir cumpliendo con sus requisitos sociales.

Zaragoza, octubre 2010

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Landslides in reservoirs and dam operation

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ABSTRACT: Risk of landslide occurrence in reservoirs is enhanced by the submergence of the slope toe and by rapid drawdown conditions. Instabilities may endanger the dam itself. They may also lead to strong wave motion in the reservoir in case of rapid landsliding. The paper reviews a few recent cases of fast landslides into reservoirs. Two case studies are also presented in more detail. The first one refers to rapid drawdown. It is concluded that a proper analysis requires a coupled flow-deformation approach. The second case (Canelles landslide) describes the cause of first instability (rapid drawdown), the validation of the hydrologic model of the landslide and the thermal analysis of a rapid motion. Recent developments indicate that permeability in the vicinity of the sliding surface is a key parameter to decide the risk of rapid motion. The paper points out also the risk of slope instability associated with excavations required for dam construction (foundations, portals of diversion tunnels, spillway, etc.). The paper provides a set of conclusions inspired in the review of case histories and in the analyses of real cases performed.

1 INTRODUCTION

In the design, construction and operation of dams and reservoir sites, instabilities of banks and ancient landslides by reservoir impoundment are one of most important issues to face. Most of dams and reservoirs are located in natural valleys frequently resulting from fluvial or glacial erosion whose banks are potentially unstable. There is a risk that engineering projects activate or reactivate landslides. In addition, it is common to find valleys whose geologic structure is a syncline involving stratification layers parallel or subparallel to the topographic slope. This situation favours the motion of landslides into reservoirs. In these cases, potential sliding planes are generally located in layers, faults or shearing zones that exhibit smaller shear strength than the surrounding strata.

There is a wide variety of slides depending on the materials involved, morphology, rate of movement, triggering mechanism and type of run-out. Several authors have worked on their definition and classification (Nemcok, 1972; Varnes, 1978; Hutchinson, 1988, among others). A European classification used by Dikay *et al.*, (1996) distinguishes between:

- *Fall*, that usually denotes the free-fall movement of material from a steep slope or cliff;
- *Topple*, which is very similar to a fall in many respects, but normally involving a pivoting action rather than complete separation at the base of the failure;
- *Slide*, in which a mass moves over a well-defined failure surface where relative displacements are concentrated;
- *Lateral spreading*, characterized by the low-angle slopes involved and its unusual form and rate of movement;
- *Flow*, that behaves as a fluidised mass in which water or air are significantly involved;
- *Complex*, which involves changes of behaviour during downslope motion. It may be described by two or more of the previously mentioned movements.

The work presented in this paper focuses on the third group of movements: slides that move on a shearing surface with a relatively light degradation of the mobilized body.

1.1 *Reservoir level and landslide risk*

Reservoir operation implies two unfavourable effects for the stability of banks and slopes:

a. The submergence of the toe of a potential landslide
b. The rapid drawdown condition

Regarding the first point, considerer in Figure 1 a slope partially submerged. The groundwater profile will be affected by the position of the reservoir water level whether the slope is fed by the stored water or viceversa. Considerer a potential failure surface and the horizontal free water level within the slope indicated in the figure. This particular case results in hydrostatic profiles of pore water

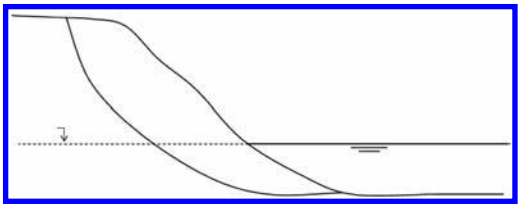


Figure 1. Slope partially submerged.

pressure inside the slope. The progressive inundation of the toe of the slope implies an increment of pore water pressure acting along the sliding surface that contributes to reduce the effective stress and therefore, to reduce the frictional strength. This reduction of effective stress is partially compensated by a total stress increment due to the load of the water weight against the slope surface. This total stress increment along the sliding surface is not easily calculated. However, given the geometry of the failure surface and the slope, it can "a priori" be estimated that pore water increment may not be completely compensated by the increase in total stress induced by water loading on the exposed surface and, therefore, a reduction of the effective stress takes place. Under these conditions, resisting forces decrease. However, this negative effect may be compensated by the beneficial increment of the hydrostatic forces acting on the exposed slope, which oppose the slide motion.

In fact, the resulting changes of safety factor, when the external level changes, depend on a number of factors, including slope and failure surface geometry. The interesting result is that a minimum safety factor is typically calculated for an intermediate water level. Consider the simple case represented in Figure 2. The three cases shown were solved by means of a commercial slope stability program for soil slopes (GEO-SLOPE 2004 developed by GEO/SLOPE International Ltd. Calgary, Alberta, Canada). The Morgenstern-Price method (Morgenstern and Price, 1965) has been used. In all cases, the distribution of pore pressure inside the slide follows a horizontal water table and no suction effects have been considered. Minimum safety factors are obtained when water level reaches a value varying between 1/3 to 1/2 of the slope height. Note that slopes with a lower planar or sub-planar failure surface (cases "a" and "c" in Fig. 2) are especially sensitive to submerging the toe if compared with a failure surface dipping in the direction of the slope itself (case "b").

The second critical situation met during reservoir operation, rapid drawdown, is a classical scenario in slope stability, which arises when totally or partially submerged slopes experience a reduction of the external water level. This is a common situation in riverbanks, subjected to changing river

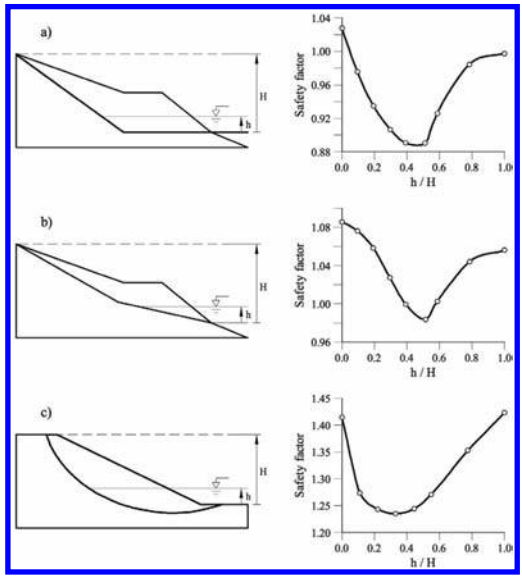


Figure 2. Evolution of safety factor (Morgenstern-Price method) when the water level in the reservoir increases. *h*: water level above the elevation of the exit point of the sliding surface. *H*: maximum value of h, when the entire slope is submerged. Case (a): geometry of Vaiont and (c' = 0; φ' = 12º); Case (b): geometry modified from Case (a) and (c' = 0; φ' = 15º); Case (c): conventional slope and circular failure surface (c' = 0; φ' = 30º).

levels. Flooding conditions are critical in this case because river levels reach peak values and the rate of decreasing water level tends to reach maximum values also.

Rapid drawdown conditions have been extensively analysed in the field of dam engineering because reservoir water levels fluctuate widely due to operational reasons. Drawdown rates of 0.1 m/day are common. Drawdown velocities of 0.5 m/day are quite significant. One meter/day and higher rates are rather exceptional. However, reverse pumping storage schemes or dam flow discharges when reservoir level is low may lead to such fast water level changes in reservoir levels.

Sherard *et al.* (1963) in their book on earth and earth-rock dams describe several upstream slope failures attributed to rapid drawdown conditions. Interestingly, in most of the reported failures the drawdown did not reach the maximum water depth but approximately half of it (from maximum reservoir elevation to approximately mid-dam level). Drawdown rates in those cases were not exceptional at all (10 or 15 cm/day). A Report on Deterioration of Dams and Reservoirs (ICOLD, 1980) reviews causes of deterioration and failures of embankment dams. Thirty-three cases of upstream slips were collected and a third of them were attributed

to an excessively rapid drawdown of the reservoir. A significant case was San Luis dam, in California (USCOLD, 1980). San Luis dam is one of the largest earthfill dams in the world (100 m high; 5500 m long; 70 million m^3 of compacted embankment). An upstream slide developed in 1981 after 14 years of successful operation of the dam because of a drawdown, which was more intense than all the previous ones. In this case, the average drawdown rate was around 0.2 m/day and the change in reservoir level reached 55 m. Lawrence Von Theen (1985) described this case.

There is a long history of contributions, which probably starts with the analysis of Casagrande (1940), with the purpose of understanding and providing prediction tools for drawdown conditions. The main purpose of those contributions has been the prediction of pore water pressures inside the slope. Geotechnical publications place also emphasis on the determination of safety factor, although this is generally a relatively straightforward exercise (if the analysis is performed under drained conditions) if the spatial and time distribution of pore water pressures are already known.

The drawdown problem is discussed in detail in Pinyol and Alonso (2008). The drawdown condition in a slope is analysed as a fully coupled flow-deformation problem for saturated/unsaturated conditions. Some fundamental concepts are first discussed in a qualitative manner and, later, explored in more detail in synthetic examples, solved under different hypothesis, including the classical approaches. Some design rules, which include a few fundamental parameters for the drawdown problem, have also been solved in a rigorous manner to illustrate the limitations of simplified procedures.

The relevance of rapid drawdown is discussed again at the end of this paper where a very large landslide (40 millions m^3 approximately) triggered by a rapid drawdown on the left margin of Canelles reservoir (Catalonia, Spain) created a situation of risk for the operation and management of the dam and reservoir area.

Dam construction requires auxiliary excavations to build foundations, diversion tunnels, spillways and other auxiliary installations (power houses). Ground stability should be analyzed and guaranteed because of the negative effects on dam construction. However, it is a common occurrence to find instabilities in dam sites. Two examples have been briefly described.

2 FIRST TIME AND ANCIENT LANDSLIDES

From a geotechnical perspective, it is important to distinguish between first-time failures and reactivation of ancient landslides. The first type develops in "intact" sites. These types of slides are difficult to analyze, especially when brittle materials are involved in the potential failure surface. It is the case of hard soils or soft argillaceous rocks, in particular those having a high plasticity, the strength operating in practice is especially difficult to predict. In fact, materials exhibit a shearing softening behaviour from a peak value, associated with a low value of shearing displacement, to a low residual strength value, reached when clay particles orient in the direction of shearing. Drained ring shearing tests and reversal direct shear tests allow the estimation of the residual strength, which is mainly controlled by particle size and the mineralogy of the soil. A simple but wrong conclusion in the analysis of first-time slides in brittle materials would be that the available strength is defined by the maximum peak strength reached when the slide starts to move. However, field evidence show that the available strength is intermediate between peak and residual conditions. Progressive failure phenomenon seems to have an important role to explain the failure mechanism. Early classical studies on this topic were published by Skempton (1967), Bjerrum (1967) and Bishop (1967, 1971). More recent contributions have been published by Cooper (1998), Potts *et al.* (1990), Dounias *et al.* (1996), Mesri and Shahien (2003), Alonso and Gens (2006) and Gens and Alonso (2006).

In contrast, in ancient landslides a pre-existing slide surface has been subjected to an increasing history of accumulated relative displacement. Hence, it is expected that on the sliding surface cohesion will be insignificant and the frictional angle will be close or equal to residual values.

It is widely accepted that ancient landslides exhibit a low safety factor close to conditions of strict equilibrium (SF = 1). ICOLD (2002) reports that in at least 75% of cases in which old landslides (active or inactive) are disturbed (by an excavation or by submerging the slide toe) slide reactivation or an increase in the rates of motion are observed. A simple explanation is that once the first slide occurred in the past it reached a new but strict equilibrium. Therefore, small changes in the resistant and destabilizing actions applied on the landslide body, such as excavation, erosion, weathering, changes in pore water distribution within the slope or even chemical changes, may induce the reactivation of the slide.

It is also a frequent observation when dealing with ancient landslides, especially in consolidated clayey strata, that a very low value of friction angle, even lower than the residual friction determined in ring shearing tests under the relevant effective normal stress, is operating in joints and shearing

zones. This low value is determined when testing directly the shearing surface and it is consistent with the value required to explain the instability by back-analysis.

Several reasons have been invoked to explain this result. A first explanation is provided by previous tectonical or morphological processes. In some situations, folding or tilting sedimentary strata having different stiffness leads to relative displacement between interbedded contacts. This idea was represented by Fell *et al.* (2005) by means of Figure 3. Residual conditions are reached at the interface between weak and hard bedding contacts.

In addition, tectonical processes may induce a level of stress higher than the present stress intensity. Since the residual shearing strength envelope is not linear with normal effective stress (Picarelli, 1990; Stark and Eid, 1994), the actual friction angle acting on the pre-existing slip surface corresponds to the minimum value previously reached, despite the lower currently applied normal stress. This concept was schematically explained by Alonso (2000) by means Figure 4 reproduced here.

Another geological process inducing the development of residual strength in weak bedding contacts is the valley relief associated with river excavation

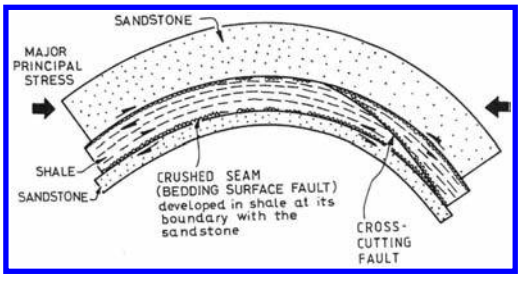


Figure 3. Usual way in which bedding surface faults are formed in mudrocks (Fell *et al*. 2005).

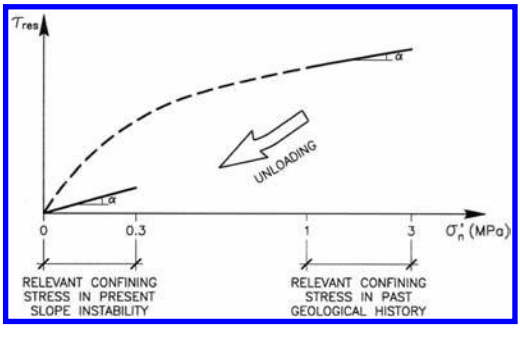


Figure 4. Residual strength envelope in residual conditions on sliding surfaces (Alonso, 2000).

that especially affects dam and reservoirs sites (Burland *et al*., 1977).

Weathering and changes in mineralogy of the material located in the slide surface may also lead to a significant reduction of the available shearing strength. In particular, sliding surfaces are prone to exhibit changes in their mineralogy whenever they become preferential paths for water flow which leads to the dissolution of salts and other minerals. Different authors have shown the negative effects of chemical alteration and weathering on the friction angle. Hawkins *et al.* (1988) observed that the loss in calcite content leads to a reduction of 25% of friction angle; Rodríguez-Ortiz and Prieto (1980) reported the effect of sulphate content when evaluating the stability of natural; Chigira (1989) noted that the oxidation of pyrite in mudstone transforming chlorite into smectite was a cause of landslide; Elnaggar and Flint (1976) highlighted the mineralogical and geomechanical changes on the shear zone of an ancient landslide as a result of groundwater interactions in a fissured claystone. (The last two works were mentioned by Hamel, 2004.)

Another interesting case is the Cortes landslide (Valencia, Spain). It was located on the left bank of Júcar River, upstream but very close of the Cortes arch-gravity dam (Alonso *et al*., 1993). The excavation of the upper part of the landslide in order to ensure its stabilization allowed the direct observation of the striated shearing zone. It was shown to be a few millimetres thick, striated layer of greenish colour in contrast with the brown colour of the surrounding marl that constituted the thicker strata where the failure surface was located. Mineralogical analyses by X-ray diffraction were made on samples of the thin greenish layer and, for comparison, on samples taken a few millimetres apart, within the thick marl layer. The main layer was described as an argillaceous dolomitic layer (Fig. 5a). However, in the thin greenish layer, the dolomite was almost absent (Fig. 5b). This change in mineralogy affected the mechanical features of the material. Direct shearing tests on the actual sliding surface aligned carefully with the middle plane of the shear box provided a friction angle equal to 17°, five degrees lower than the residual friction obtained in reconstituted clayey dolomitic samples tested in the ring shear test.

The mentioned considerations about the strength available along the slide surface are important because they make it necessary to test the strength on intact samples taken on the sliding surface. In the cases mentioned before, the usual ring shear test on reconstituted samples of the material around the sliding surface will overestimate the residual strength.

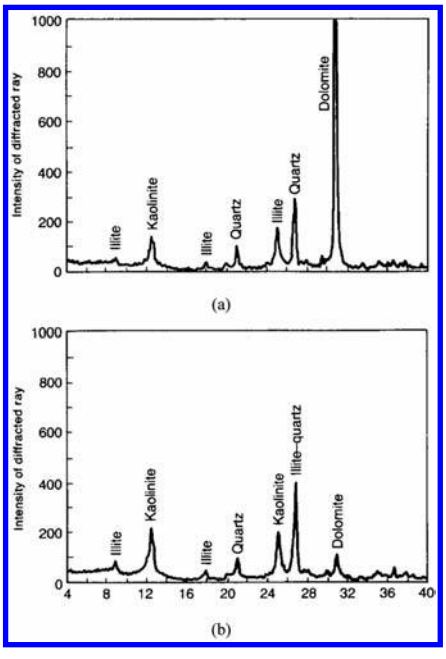


Figure 5. Difractograms of two neighbouring samples within the marl layer of Cortes slide: (a) main body of the layer, brown and beige; (b) striated sliding surface, grey and green (θ = angle of diffracted ray) (Alonso *et al*., 1993).

3 CONSEQUENCES OF LANDSLIDES IN RESERVOIR SITES

In general, slope instabilities imply a risk that should be prevented for its dangerous consequences: damage in roads, buildings, foundations, etc. However, slope instabilities around reservoirs, being them induced or not by the stored water, include additional risks:

- Damage to the dam or its foundation.
- Partial or complete blockage of the storage water and the formation of natural dams that induce a reduction of the volume of the reservoir and a non controlled increase in the upstream water level. The failure of such dams can be disastrous because of the risk of dam overtopping.
- Depending on their velocity, they can generate a destructive wave when the landslide hits the reservoir water. The wave will propagate and may induce a dam failure or a dam overtopping.

4 FAST LANDSLIDES

The risk associated with landslides in reservoirs attracted wide attention in 1963 when the catastrophic failure of the left bank slope in Vaiont reservoir in Italy caused more than 2000 deaths and a mayor destruction (Hedron and Patton, 1985; Müller, 1964). The case is reviewed often in the literature mainly because of the difficulty of determining the causes of its great acceleration. The slide reached a velocity around 25–30 m/s (100 km/h), an estimation based on the height of the ensuing wave.

The risk of landslide acceleration may become one of the most challenging problems to face for people in charge of reservoir and dam operation. The question of what speed can reach a specific landslide and its consequences does not have an easy response. The analysis of the whole phenomena requires assessing the following phenomena (Fig. 6):

- Triggering of the landslide.
- Run-out.
- Impact of the landslide against the reservoir and generation of a destructive wave.
- Impact on dam and facilities around the reservoir.

Most of the models developed so far to describe the wave generation and propagation take into account the hydro-dynamic of the water body, making assumptions on the falling mass, although they do not analyze in detail the landslide run-out (Harbitz *et al*., 1993; Noda, 1979; Biscarini, 2010; Capone, 2009; Walder *et al*., 2003). Pastor *et al*. (2009) presented a model able to describe all mentioned phases, from landslide triggering to wave propagation, considering the coupled interaction between the avalanche of a fluidized soil and the water body.

The risk of a landslide reaching a high velocity depends basically on the differences between resistant and destabilizing actions applied on the unstable body. There are different factors that may lead to the acceleration of landslides:

- Resistant actions (shear strength) decrease due to strength softening (typically in brittle materials) and to rate effects on residual strength.

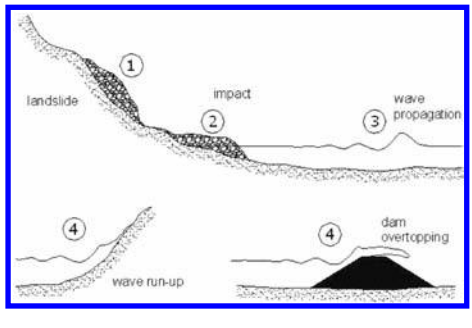


Figure 6. Principal phases in the phenomenon of landslide-induced wave and consequences (Capone, 2009).

– Generation of compressive pore water pressures on saturated sliding surfaces in undrained o partially drained conditions. It typically occurs in loose granular soils (which are prone to flow liquefaction).
– Generation of water pressure due to fragmentation or crushing of particles formed by shearing in granular materials (Sassa *et al*., 1996; Wang and Sassa; 2002).
– Pore water pressure generation by thermal effects due to frictional shearing at the shear band.

The discussion on the basic phenomena leading to the fast acceleration of a landslide is today very active and each case should be analysed separately.

A number of case histories will be briefly presented. They illustrate the magnitude and dangerous consequences of landslides that accelerate and invade reservoirs and rivers. The occurrence of such events is not frequent and there is a lack of well-documented case histories. Most of the cases described below may be criticized for being too descriptive but they strongly convey the risk of these rare events.

5 CASE HISTORIES FROM LITERATURE

In this Section, the causes, features and consequences of landslides around reservoirs are exemplified by means of the description of some published real cases.

Two important cases of non-catastrophic landslides in Spain associated with reservoirs, Cortes landslide (Valencia) and Canelles landslide (Catalonia) are not mentioned here. Cortes landslide is analysed in Alonso *et al.* (1993). Canelles landslide is currently a potentially dangerous case for the operation of the reservoir. The landslide was first observed in summer 2006. The case is presented in the last part of this paper.

In the following description of catastrophic landslides, the most famous and well-documented Vaiont landslide has been recently described in detail in Alonso and Pinyol (2010) and Pinyol and Alonso (2010).

The landslides collected here are divided in two groups: those which may rationally be viewed as potentially dangerous and those which actually failed at high speed and resulted in a catastrophic situation.

5.1 *Potentially dangerous landslides*

5.1.1 *Aknes landslide, Norway*

Aknes is a large rockslide continuously creeping located above a fjord in Norway, in the vicinity of several communities and tourist places (Ganerød *et al.*, 2008) with an estimated volume of 35–40 millions of m^3 and dimensions of 800 m across-slope and 1000 m down slope (Fig. 7).

The unstable mass is defined by a back scarp, a basal shear zone, 50 m deep and a toe zone where the basal sliding surface daylights on the slope surface. Figure 8 shows the geological model of the Ankes rockslide presented by Ganerød *et al.* (2008). It was obtained by geological surveys using different techniques (structural maps, geomorphologic interpretation, penetrating radar, seismic refraction, 2D resistivity profiling and sonic logging of p-waves). The rockslide can be divided

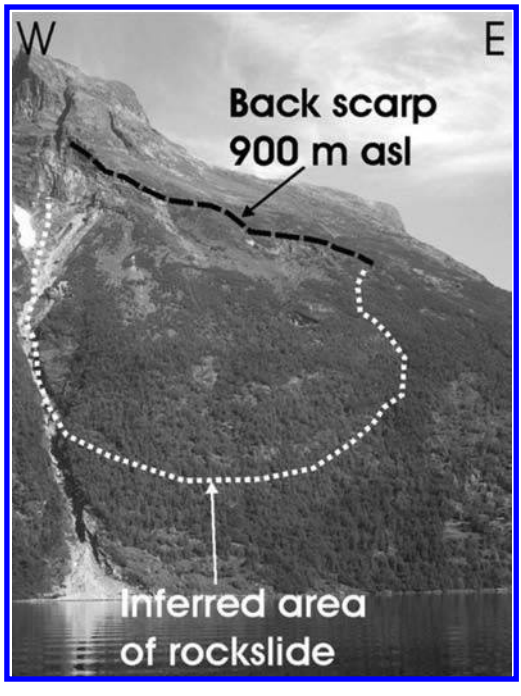


Figure 7. Overview of Aknes landslides over the fjord in Norway (Ganerød *et al.,* 2008).

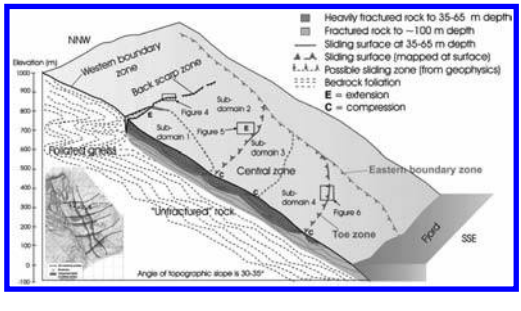


Figure 8. Geological model of Aknes landslides (Ganerød *et al*., 2008).

into an upper part (subdomains 1 and 2, Fig. 8) that experience extension, and a lower part (subdomains 3 and 4) that deforms by compression. The upper boundary zone of the rockslide is seen as a back scarp that is controlled by, and it is parallel to, the pre-existing steep foliation planes of gneiss. The foliation in the lower part seems to control the development of the basal sliding surface which daylights at different levels. The sliding surfaces are subparallel to the topographic slope and are located along mica-rich layers in the foliation (Ganerød *et al.,* 2008).

The shear strength of the material involved in the sliding surface of Aknes landslides has been reported by Grøneng *et al*. (2009). Samples from gouge material, unfilled rock joints and intact rock were tested. Gouge material (9–19% clay content) tested in triaxial test exhibited a friction angle between 18 and 35° depending on the mineralogy. Samples containing the lowest percentages of quartz and highest percentage of mica show the lowest shear strength. Rock joints shear strength was evaluated by empirical methods (Barton and Choubey, 1978) resulting in a friction angle of 24°. The tested intact gneiss showed a shearing strength defined by 19–25 MPa of cohesion and 57–89° of friction angle.

5.1.2 *Maoping landslide, China*

Maoping landslide is the largest ancient landslide located on the left bank of the Qingjiang river (one of the major tributaries of the Yangtze river). It is located 66 km upstream of Geheyan dam and it was reactivated by the impoundment of the reservoir (2.9 billions m^3) in April 1993.

The Maoping landslide was geologically investigated before reservoir inundation and its displacements are being controlled and correlated with reservoir level and rainfall. A total of 34 landslides (in total 0.16 billions of m^3 in volume) were identified on both banks of the reservoir.

The reactivation of the Maoping landslide required the relocation of a small village located on the valley slope. Qi *et al.* (2006) the main source for the summary presented here. The volume of the landslide is estimated in 23.5 millions m^3. Figure 9 shows an overview of the landslide. The thickness of the slides varies from 5 to 89 m, increasing from the rear to the front and from the west to the east.

The mobilized mass includes three main layers. The first layer is constituted by a detritic soil (10–20 m thick) occasionally containing limestone blocks. The second layer (5–15 m thick) is constituted by quartz sandstone blocks, often overlaid by a layer of 2–8 m thick of coal soil and gray white detritus and fragments. The third layer (20–40 m) is bedrock-like disintegrated blocks of sandstone. The sliding plane was developed along a bedding surface in the sandstone, which is inter layered by shales with a dipping angle of 15–20°, except for the front part (having a length of 180 m) where it is near horizontal. However the sliding zone has a different appearance when one compares the west and eastern parts. This difference seems to be the result of different sliding processes and sliding sequences in the history of the landslide. The formation process of the Maoping landslide was a result of three major landslides shown in Figure 10. The landslide previous history is complex but the fundamental information is that it is a reactivation of an ancient landslide.

Stability analyses based on strength parameters obtained in the laboratory (Qi *et al*., 2006) were carried out before and during the reservoir impoundment. Drained strength parameters were obtained in consolidated quick tests on saturated samples which gives a doubtful reliability to the results.

Cracks and settlements were first observed in the ground and houses after reservoir impoundment. During the rain seasons, cracks opened. According to observations the maximum accumulated horizontal displacement of the slide reached 2840 mm, in the period March, 1993 to October, 2005.

Figure 9. Overview of Maoping landslide (Qi *et al.,* 2006).

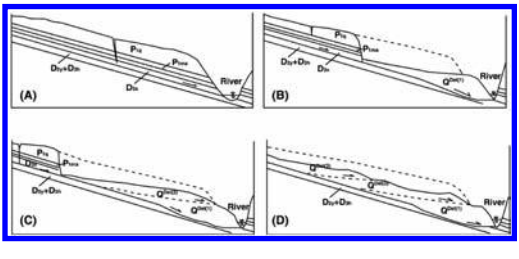


Figure 10. Formation process of the Maoping landslide (after Bureau of investigation and survey, Changjiang water resources commission, 1993. From Qi *et al*., 2006).

5.2 *Catastrophic landslides*

5.2.1 *San Juan de Grijalva landslide, Mexico, 2007*

In November 4, 2007, a catastrophic landslide took place in the left margin of the Grijalva river located in the south-eastern province of Chiapas, Mexico. It was a translational movement with a length of 1200 m, 610 m wide and a depth of 70 m. The volume was around 50 million m^3. The run-out of the landslide was about 800 m.

The impact of the slide against the water of the river generated a wave which was reported to be 50 m high (Alcántara-Ayala *et al*., 2008) or 15 m high (Hernández-Madrigal *et al*., 2010) depending on the source. The landslide-generated wave destroyed the San Juan de Grijalva village. A lake was formed due to a landslide-induced dam, which flooded 21 villages upstream. The landslide and its subsequent consequences killed nineteen people and six more were reported missing.

The landslide was registered in a seismic station located at a distance of 16 km. Such records indicated that the mass movement took place during approximately 80 seconds (Alcántara-Ayala *et al*., 2008). Before the slide, the topography of the valley was characterized by a 5–20° slope on the upper hillside, and a 20–40° slope on the adjacent right bank of the Grijalva River. At first sight, such geometry of the sliding surface leads to the deceleration of the landslide.

A 80 m high and 1170 m wide dam was created across the Grijalva River. This dam created a lake having a surface of 49 km^2. Figure 11 shows a photograph of the natural dam induced by the landslide.

The geological setting includes alternating conglomerates, sands, silts, clays and marls of Eocene and Miocene age, which overlie Jurassic and Cretaceous rocks (Hernández-Madrigal *et al*., 2010) (Fig. 12). Alcántara-Ayala *et al*. (2008) mentioned that the slide surface was located on a clay strata dipping towards the river.

Three factors are invoked as triggering causes for the landslide (Hernández-Madrigal *et al*., 2010): the intense precipitation that took place in the previous months (the cumulative rainfall in the preceding 30 days was 67% of the average annual rainfall), an earthquake which occurred 5 days before the landslide, and the water level drawdown of the Grijalva River due to the release of water from the Peñitas dam located 14 km downstream. Figure 13 shows the water level at the Peñitas reservoir (which controls the level of the river upstream). The landslide occurred almost at the end of the final drawdown. The drawdown velocity at the end of drawdown period was 1.47 m/day, 4.2 times faster than the average drawdown rate of 0.35 m/day.

Figure 11. San Juan de Grijalva landslide blocking the river (Alcántara-Ayala *et al.,* 2008).

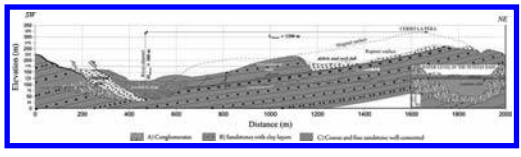


Figure 12. Geological cross-section of San Juan Grijalva landslide (Hernández-Madrigal *et al.,* 2010).

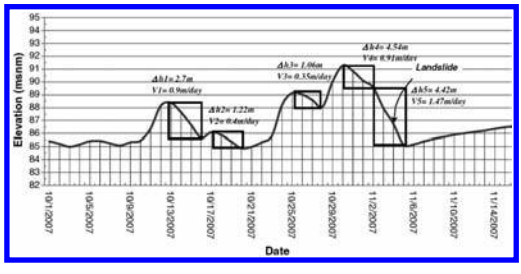


Figure 13. Water level at the Peñitas dam before San Juan de Grijalva landslide occurred in November 4, 2007 (Hernández-Madrigal *et al.,* 2010).

A channel was excavated in the natural dam to restore partially the natural flow conditions of Grijalva river.

5.2.2 *Qianjiangping landslide, China (2003)*

Qianjiangping landslide occurred after the first impoundment of the Three Gorges Reservoir in 14th July, 2003. It was located in the left bank of the Qinggan River, a tributary of the Yangtze River. It happened when the reservoir reached a water height of 135 m. Erosion by the river, quarrying of shale in the landslide toe and previous heavy rain seem to be additional combined causes leading to the instability (Wang *et al*., 2004).

Figure 14 shows the landslide form the upstream side and a frontal view of the landslide.

The landslide had a tongue-like shape and has a length of 1200 m and a width of 1000 m. The average thickness of the sliding mass was about 20 m, thinner in the upper part and thicker in lower part. The total volume was estimated to be more than 20 millions of m^3. It displaced forward around 250 m. Figure 15 shows a representative section of the slide. It was a typical translational rockslide.

The landslide caused loss of life and serious damage with important economical cost. Thirteen people lost their lives on the slope banks and eleven fishermen died while working on boats in the nearby area. The casualties on the river were a consequence of the high velocity of the slide which triggered a 30 m high wave. Witness accounts showed that it took few minutes to complete the whole sliding process (Wang *et al*., 2004). The slide velocity was not given despite the available data on the wave height.

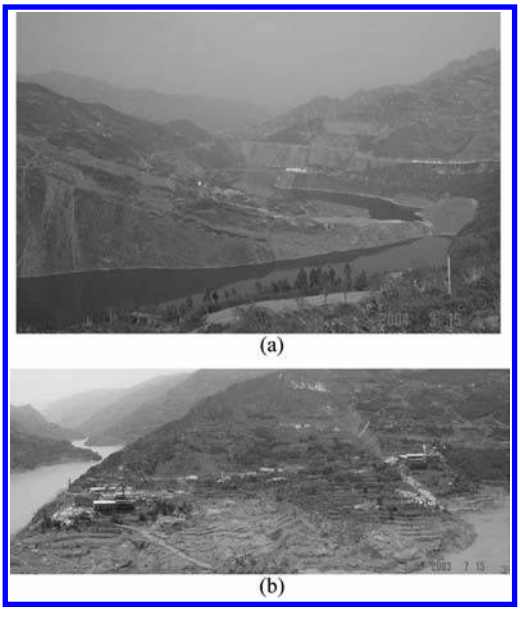


Figure 14. View of the Qianjiangping landslide: (a) from the upstream side; (b) front view (Wang *et al.,* 2004).

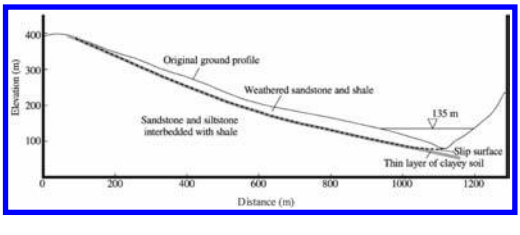


Figure 15. Qianjiangping landslide representative section (Dai *et al.,* 2004).

The landslide completely blocked the Qinggan River and formed a lake upstream. To prevent that the water in the impounding area reached a dangerous level with possible damage downstream in case of dam breaching, a channel diversion was excavated (Dai *et al*., 2004).

The displaced block was constituted by layers of weathered sandstone and shale. According to Dai *et al.* (2004), the slip surface was located in a thin layer of clayey soil having a thickness of several centimetres. The bedrock below the sliding surface was constituted by sandstone and siltstone layers interbedded with intact shale.

The sliding surface could be uncovered in the upper part of the landslide (Wang *et al.*, 2004). According with their interpretation, Figure 16a shows the original situation, previous to the 2003 landslide. The photograph 16a shows a striated surface which indicates the initial sliding direction (red arrows). Figure 16b corresponds to new striations that follows the sliding direction of the 2003 landslide (indicated by the blue arrow). The pre-existing shearing surface is associated with an older geological event (probably during intense folding in a Cretaceous Period) (Wang *et al*., 2004). The existence of widely distributed striation on the sliding surface as well as other geomorphological indications leads to accept that the sliding surface was mobilized prior to the landslide in 2003. However, since the "old" and "new" directions of sliding are not the same, it would be interesting to evaluate if the shearing strength along the mobilized shear surface remains in residual conditions when the shearing direction changes.

A more detailed exploration of the sliding zone is presented by Wang *et al.* (2008). Two horizontal tunnels were excavated with the purpose of reaching the sliding zone. Two different materials were found: a layer of black silt with calcite blocks (w_L = 30%; PI = 2) on the top, while the lower strata was a yellow clay (w_L = 49%; PI = 40). Figure 17 shows a photograph of the sliding zone

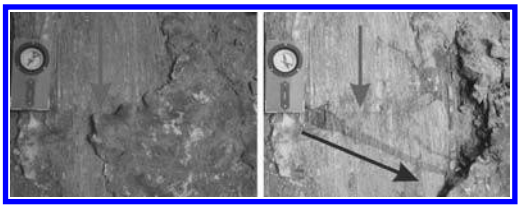


Figure 16. Photographs from the same site of the upper part of the Qianjiangping landslide sliding surface showing the pre-existing scratches that underlie the sliding surface of the July 2003 event (blue arrow shows the sliding direction, red arrows show the strike direction of the pre-existing scratches) (Wang *et al*., 2004).

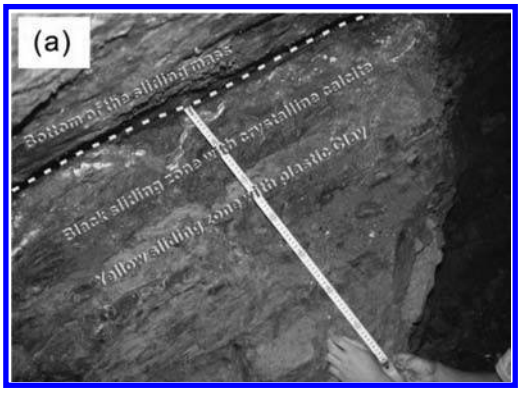


Figure 17. Photographs of sliding zone of Qianjiangping landslide taken from horizontal tunnel (Wang *et al*., 2004).

taken in one of the horizontal investigation tunnels. Samples of the two types of materials were tested by direct shearing imposing different rates of displacement (Wang *et al.*, 2008). A mobilized residual friction angles (reached after more than 2000 mm of relative displacement) equal to 26° and 20° were obtained for the reconstituted black silt and the yellow clay samples, respectively. Undrained rapid ring shear tests indicated that a rate of displacement around 10 mm/sec induces a drop of the strength of around 10° on the black silty soil. The rate of shearing did not affect the yellow clay.

A fundamental question rises from the description of Qianjiangping landslide: ¿What was the cause of the rapid loss of shearing resistance necessary to achieve the high rate of acceleration? The representative cross-section shown in Figure 15 does not explain the development of a high velocity. Several alternative reasons could be possibly advanced, but an accurate analysis and further testing may be required. A drop of strength as the displacement increases could explain such acceleration. However, if it is true than the sliding surface is in residual conditions, the loss of strength with displacement increment is unlikely. Wang *et al.* (2004) mentioned the presence of crystalline calcite on the sliding zone, acting as a cement deposited after the folding of sedimentary strata that mobilized the sliding surface. Crystalline calcite is characterized by brittle facture and this phenomenon could explain the sudden loss of strength.

A more complex mechanism is described by Wang *et al.* (2008). They explain that the effect of shearing rate is the actual reason for the strength softening behaviour. None of the authors considers the effect of heating induced by frictional work at the sliding surface. This approach explains the build-up of pore pressure by the movement of the slide. The shearing displacement generates frictional work that dissipates in heat. Temperature increments lead to the dilatation of solid particles and water that, in a saturated porous media, and always depending on properties such as stiffness and permeability, leads to pore water pressure build-up. As a result, the effective normal strength reduces and, accordingly, the available frictional strength. Checking this hypothesis requires a specific thermo-hydro-mechanical analysis of the landslide.

5.2.3 *La Josefina landslide, Ecuador (1993)*

On March 29, 1993, a massive rockslide dammed the Río Paute in Cuenca, Ecuador. It situation is shown in Figure 18. The slide was 1500 m long and 600 m wide. The volume of the moving mass was estimated to be around 20 million m^3 (Harden, 2004) of igneous rocks overlain by colluvial deposits.

Aerial photographs taken before the failure and the observation of a headscarp approximately 80 m high, led to the conclusion that the movement was a reactivation of an ancient landslide. It was probably triggered by heavy rain and mine excavations at the toe.

The regions upstream and downstream from the landslide were densely populated. Seventy two people were reported killed or missing (Chamot, 1993) and the economic losses were devastating.

The slide formed a 100 m high natural dam on the Río Paute at its junction with the Río Jadan (Fig. 19). The impoundment behind this dam flooded the upstream valley for a length of 10 km, submerging agricultural land, houses and industries. The final stored water volume of the natural dam was 200 million m^3. The lake reached a depth of 83 m. After 33 days, the dam failed, resulting in a peak discharge of 10000 m^3/s. The resulting debris flow and mudflow flooded the valley downstream entering into the Amaluza Reservoir behind the dam with the same name. Reservoir authorities manage to lower 31 m the reservoir level. In spite of this precaution, severe damage on the powerhouse turbines occurred

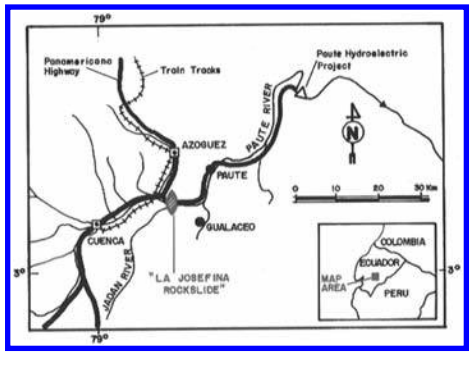


Figure 18. Location map for La Josefina rockslide (Plaza-Netos and Zevallos, 1994).

due to the high concentration of suspended soils in the water and also on houses and industries located around the affected site (Schuster *et al.,* 2002).

The failure of the landslide-induced dam led to a decrease in the water level of the Josefina lake at a relatively high velocity and several landslides occurred on the surrounding slopes due to the rapid drawdown. The most important one was Zhizhio slide (Schuster *et al.,* 2002). No additional information on theses landslides has been found.

5.2.4 *Gros Ventre landslide, United States (1925)*
The Gros Ventre landslide is located in Bridger-Teton National Forest, Wyoming, U.S.A. The landslide occurred on June 23, 1925, after several weeks of heavy rain. The landslide created a huge dam across the Gros Ventre River, which stored the water and formed a lake. Approximately 38 millions m³ of primarily sedimentary rock slid down the north face of Sheep Mountain, crossed over the Gros Ventre river and raced up the opposing mountainside a distance of 100 m (Fig. 20). The landslide created a large dam over 60 m high and 400 m wide across the Gros Ventre River. On May 18 1927, a portion of the landslide dam failed, resulting in a massive flood that reached a height of 2 m 40 km downstream. The small town of Kelly, 10 km downstream, was destroyed and six people were killed.

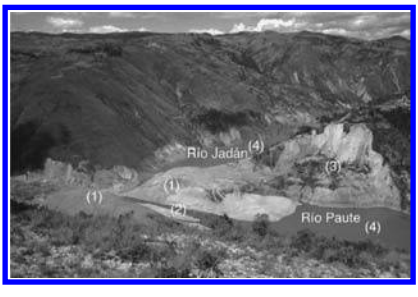


Figure 19. The landslide scar and dam in 1994, 1 year after the dam was ruptured. The photograph was taken from near the top of the slide, looking southward; the flow of the Paute River is from right (west) to left (east). Numbered features are (1) deposits from the March 29, 1993 landslide, (2) channel excavated to release impounded water, (3) a post-rupture slope failure at Zhizhio, and (4) remnant lakes a year after the dam rupture (photo by Harden, 2004).

Figure 20. Gros Ventre landslide overview (Shelton, 1966).

6 CASE STUDY I: SHIRA AND SAN SALVADOR DAM

This case refers to rapid drawdown conditions and its effect on dam stability. The key aspect in design is the correct prediction of pore water pressures. The difficulty to make predictions by simple methods of analysis will be highlighted by comparing the reaction of the two dams.

Drawdown conditions are often solved by flow calculations, which implicitly assume a rigid soil skeleton. In this case study pure flow estimations and the results of a correct approach: a fully coupled flow-deformation analysis (which should account also for unsaturated conditions) are compared.

The dam cross section is schematically indicated in Figure 21. Parameters for the analysis are given in Table 1. They were determined from tests performed at the design stage of the dam.

Figure 21 shows a comparison of calculated pore water pressures alter drawdown for the coupled and uncoupled cases. The analyzed drawdown corresponds to the design specifications: reservoir level decreases 24 m in 60 days. Calculated pore pressures in the upstream shell, core and foundation under the hypothesis of uncoupled analysis are significantly higher than in the coupled case.

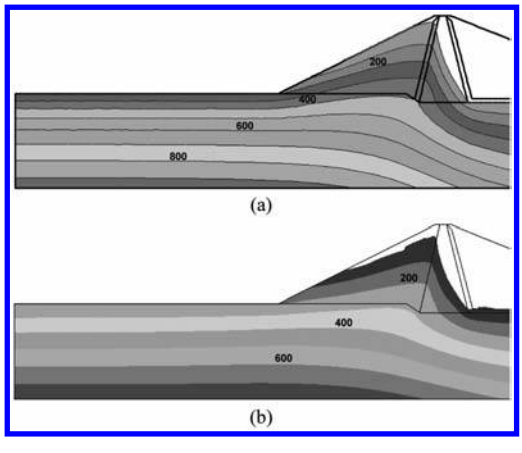


Figure 21. Pore water pressure contours. The represented interval is 100 kPa. (a) Uncoupled analysis. (b) Flow-deformation coupled analysis.

Table 1. Parameters for the drawdown analysis of San Salvador dam.

Parameter	Symbol	Unit	Foundation	Upstream shell	Core
Young modulus	E	MPa	150	100	30
Coefficient of volumetric compressibility	m_w	MPa^{-1}	$4.95 \cdot 10^{-3}$	$7.42 \cdot 10^{-3}$	$2.47 \cdot 10^{-2}$
Saturated permeability	k_{sat}	m/s	$1 \cdot 10^{-9}$	$1.8 \cdot 10^{-9}$	$2.81 \cdot 10^{-10}$
Retention curve (Van Genuchten)	P_0	MPa	0.5	0.05	0.5
	λ	–	0.24	0.4	0.24
	S_{rmax}	–	1	1	1
	S_{rmin}	–	0.01	0.075	0.01

This is clear also in Figures 22 to 24 which provide the evolution of pore water pressures in three representative points of the dam: two in the foundation and a third one in the shell, close to the core. Figure 22 indicates that non-coupled analyses are unable to reproduce an elementary result: pore pressures under the bottom of the reservoir should follow, in an essentially instantaneous manner, the variations of reservoir water level. The uncoupled analysis results in pore pressures higher than the level in the reservoir.

A similar result is observed in a profile directly affected by the dam (below the upstream toe; see Fig. 25). Three cases are represented: initial profile, profile immediately after drawdown and long term. In the correct coupled analysis pore pressures after drawdown are higher than the hydrostatic long term values. This is due to the presence of the dam and the particular stress distribution associated with changes in total stresses against the boundary of the dam and the foundation soil. Pure flow analysis results in abnormally high pore water pressures.

Consider now the second case Glen Shira Lower Dam which is part of a pumping storage scheme in Northern Scotland. The reservoir was expected to experience fast drawdown rates and this situation prompted the field experience reported by Paton and Semple (1961). Probably this is one of the best-documented case histories concerning the effect of drawdown on earth dams. The maximum cross section of the dam is presented in Figure 15. The 16 m high embankment has a centered thin reinforced concrete wall. The homogeneous embankment is made of compacted moraine soil. A rockfill shell covers the upstream slope of the compacted moraine to increase stability. Published grain size distributions of the moraine soil indicate a well-graded material having a maximum size of 15 cm. Plasticity is not reported for this soil. It was apparently compacted wet of optimum at an average water content $w = 15\%$. The attained average dry density was 19.8 kN/m^3, which is a relatively high value for a granular mixture. A friction angle $\phi' = 36°$ is reported.

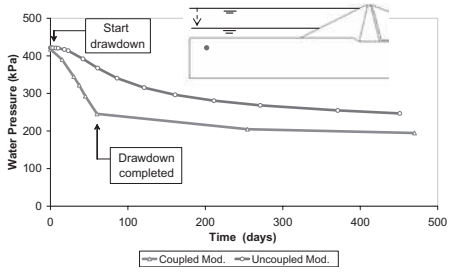


Figure 22. San Salvador dam. Evolution of pore pressures in a point distant from the dam toe, during drawdown and subsequent times.

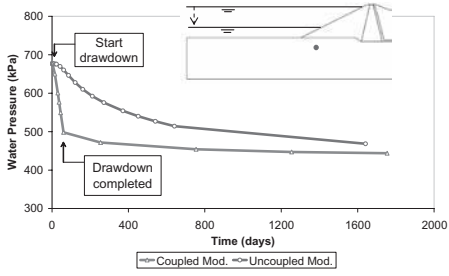


Figure 23. San Salvador dam. Evolution of pore pressures in a point within the foundation, Ander the upstream shell during drawdown and subsequent times.

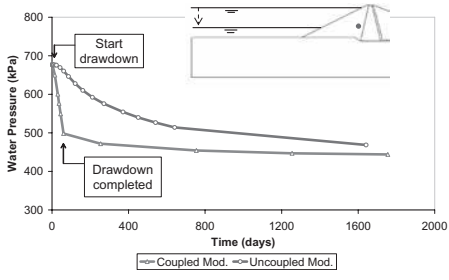


Figure 24. San Salvador dam. Evolution of pore pressures in a point within the upstream shell, close to the core, during drawdown and subsequent times.

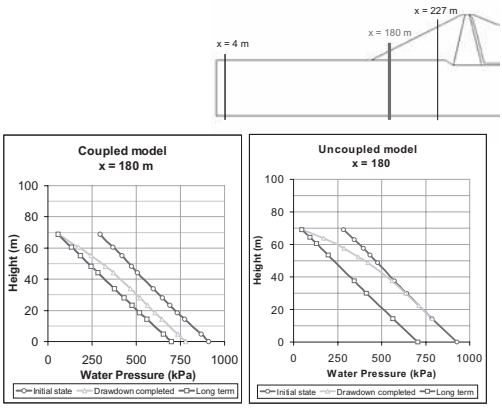


Figure 25. San Salvador dam. Vertical profiles at the toe of the dam. Comparison of coupled and uncoupled analyses.

For the rockfill a porosity of $n = 0.4$, a dry density of 16.7 kN/m^3 and a friction angle $\phi' = 45°$ are mentioned in the paper.

Five porous stone piezometer disks, previously calibrated against mercury columns, were located in the places shown in Figure 26. They were connected to Bourdon gauges through thin polyethylene tubing. The authors conclude in their paper that the possibility of instrumental error are "of minor order and can be neglected".

No significant pore water pressures were recorded during construction. Positive pore pressures were measured only after reservoir filling.

A total water level drawdown of 9.1 meters in four days was applied to Glen Shira dam. This maximum drawdown was imposed in four stages of rapid (7.2 m/day) water lowering followed by short periods of constant water level. Details of changing water level in the reservoir and the measured pore water pressures are indicated in the set of figures prepared to analyze this case.

The following hypotheses, ordered in the sense of increasing complexity, were made to perform calculations:

1. A pure flow analysis for saturated/unsaturated conditions that follows the changing hydraulic boundary conditions actually applied to the upstream slope. Table 2 provides the hydraulic parameters used in calculations. These parameters are common to the remaining analyses described below.
2. An instantaneous drawdown of the maximum intensity, followed by pore water pressure dissipation. This is a coupled analysis, which attempts to reproduce the classical hypothesis behind the undrained methods. The procedure does not correspond strictly to Bishop's method because in the analyses reported here the correct change in total stresses is actually applied. The soil was simulated as an elastic material.
3. A coupled analysis (saturated/unsaturated), following the applied upstream changes in hydrostatic pore pressures. The soil is considered elastic.
4. A coupled analysis (saturated/unsaturated) following the applied upstream changes in hydrostatic pore pressures. The soil is considered elasto-plastic following the BBM model, Alonso *et al.* (1990). The elastic parameters of this model are taken from the previous elastic model.

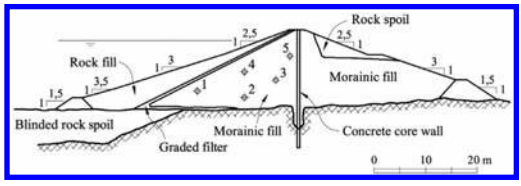


Figure 26. Maximum cross section of Shira dam. The position of piezometers 1 to 5 is indicated.

Table 2. Hydraulic parameters used for the analysis of Shira dam.

Definition of parameter	Symbol	Units	Type of soil	
			Moraine	Rockfill
Saturated permeability	k_{sat}	m/s	$1.6 \cdot 10^{-8}$	$1.0 \cdot 10^{-4}$
Relative permeability	k_{rel}	–	$k_{sat}\,(S_w)^3$	$k_{sat}\,(S_w)^3$
Van Genuchten parameter describing air entry value	P_0	MPa	0.05	0.01
Van Genuchten parameter describing mid slope of retention curve	λ	–	0.2	0.4

The case of Shira dam is especially interesting because the permeability of the compacted moraine fill (around 10^{-8} m/s; see below) is an intermediate value between impervious clay and a free draining material. One may wonder to what extent the classical hypothesis for drawdown analysis (undrained or pure flow) approximates the actual behaviour. This aspect will be discussed later.

Figures 27 and 28 illustrate the performance of the different methods of analysis (1. to 4.) listed above. Consider first the hypothesis of instantaneous drawdown (9.5 m of water level drawdown,

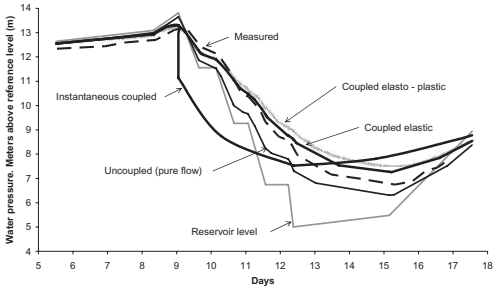


Figure 27. Comparison of measured pore pressures in Piezometer 2 and different calculation procedures.

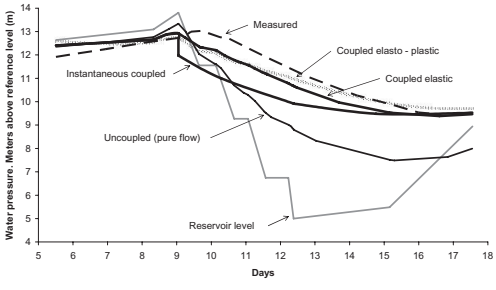


Figure 28. Comparison of measured pore pressures in Piezometer 3 and different calculation procedures.

instantaneously). The calculated pressure drop is indicated in the figures by means of a vertical bar. A (coupled) dissipation process is then calculated and the progressive decay in pore pressures is also plotted. If compared with the actual pore pressures measured at the end of the real drawdown period, the hypothesis of instantaneous drawdown leads obviously to an extremely pessimistic and unrealistic situation. (The end point of the instantaneous drawdown at $t = 9$ days is to be compared with the pore pressure recorded at the end of the drawdown period at $t = 12.4$ days).

It is also interesting to compare the results of the fully coupled analysis of the instantaneous drawdown with the approximated method of analysis suggested by Skempton/Bishop. Table 3 shows the comparison. The change in vertical stress ($\Delta\sigma_v$) has two contributions: the change in free water elevation above a given point and the decrease in total specific weight of the rockfill material covering the moraine shell. An effective saturated porosity of 0.3, after drainage, was assumed to calculate the drop in total specific weight. Bishop hypothesis leads systematically to a higher pore pressure drop than the more accurate analysis. This is specially the case for the piezometers located deep inside the fill. Discrepancies are due to the simplified stress distribution assumed in the approximate method.

Table 3. Shira dam. Instantaneous drawdown. Comparison of coupled and simplified (Bishop) analysis.

Piezo meter	Initial pressure (horizontal water table) (kPa)	Calculated instantaneous pressure drop (Code_Bright) (kPa)	Bishop hypothesis $\Delta u = B\,\Delta\sigma_V$ $B = 1\,(kPa)$
1	96	42	42
2	106	22	12
3	67	10	1
4	56	17	12
5	23	6	0

Consider now the opposite calculation method: a pure flow analysis. In this case, Figures 27 and 28 indicate that the predicted pore pressures are the lowest ones if compared with the remaining methods of analysis. Calculated water pressures follow closely the history of reservoir levels. The "damping" effect associated with soil compressibility is absent. When the water level is increased, at the end of the drawdown test, the pure flow analysis indicates, against the observed behaviour, a fast recovery of pore pressures within the embankment.

Coupled analyses are closer to actual measurements. This is true in absolute terms but also in the trends observed when boundary conditions (changes in reservoir level) are modified.

Paton and Semple (1961) plotted also contours of piezometric head during drawdown. One example is given in Figure 29. It corresponds to drawdown drop of 8.8 m. The reservoir level reaches 5.2 m (with respect to the zero reference level which is placed at the dam base). The authors used the data recorded on the five piezometers to interpolate the curves shown in the figure. They made the hypothesis of a zero water pressure at the shell-rockfill interphase. The computed distribution of heads inside the dam shell, for the same amount of drawdown, is also plotted in the figure (coupled elastic analysis). The agreement is quite acceptable, although some discrepancies exist, which, in part could be attributed to the limited accuracy of the interpolation made.

The conclusion, for the particular embankment material of Shira dam and its overall geometry and design, is that the classical methods of analysis are far from explaining the recorded behaviour. The "instantaneous" or undrained method is conservative, but very unrealistic. A fully coupled analysis of the instantaneous drawdown results in higher pressure drops than the classical Bishop proposal. At the opposite extreme, the pure flow analysis leads to a systematic and unsafe underestimation of fill pressures during drawdown. Coupled

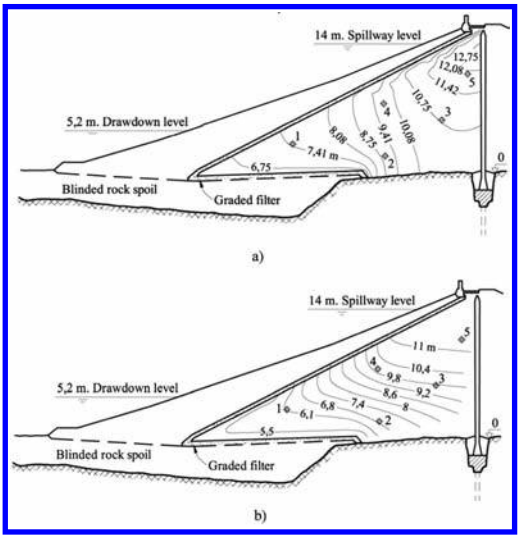


Figure 29. Distribution of pore pressures inside the shell for a drawdown 14 to 5.2 m. a) Computed results (coupled analysis); b) Interpolated values plotted by Paton and Semple (1961).

analysis captures well the actual measurements. In the case of Shira dam, plastification during drawdown was probably nonexistent, and the simpler elastic approach provides a good approximation to recorded pore water pressures.

If the cases of San Salvador and Shira dam are compared one observes that the flow analysis overestimates pore pressures in the case of San Salvador (safe but unrealistic) and underestimates pore pressure in the case of Shira dam (unsafe and unrealistic). The lower coefficient of permeability in San Salvador if compared with Shira dam explains in part the opposite trends. But the geometry of the dam, the stiffness of materials and the rate of drawdown play also a significant role. The conclusion is that simplified methods are uncertain and, it is difficult to predict at a design stage if they are safe or unsafe. The drawdown condition is a case which requires a joint flow deformation analysis for a consistent prediction of pore water pressure.

7 CASE STUDY II: CANELLES LANDSLIDE

7.1 *Introduction and background*

During the summer of 2006, after a rapid drawdown (the drawdown velocity exceeded to 1 m/day), a very large landslide developed in the left margin of Canelles reservoir (Catalonia, Spain). Canelles dam, located in the Noguera Ribagorçana river, is 150 m high arch dam.

Figure 30. Detail of the tensile crack at the foot of the jump limiting the southern boundary of the landslide. The lack of deformation, due, for instance, to surface tilting of the displaced soil (left to the crack), suggests an essentially translational movement. The remnants of the old village of Blancafort can be seen in the background of the photograph.

The geometry of the slide could be established from the detailed analysis of the continuous cores recovered in deep borings and from the limited information provided by inclinometers. Deep piezometric records provided also valuable information on the pressure changes in the vicinity of the failure surface. These data, available for a significant time, allowed validating a flow-deformation coupled calculation model that took into account the changes in water level of the reservoir and external contributions (rainfall). The model indicates that the most likely reason for failure is the rapid drawdown that took place during the summer of 2006.

An analysis of the reservoir level evolution indicated high values of drawdown velocities in the period July–August 2006. In fact, during the first days of August 2006, the velocity was greater than 1 m/day reaching a maximum value of 1.20 m/day. It is quite likely that the motion that caused the crack took place around this date. Water level elevation reduced from 440 m to 430 m. Never before, from 1986 to 2009, had drawdown velocities reached such high levels and neither had the water level elevation been so low.

Geomorphological evidence indicates that the slide is not a first failure, but the reactivation of a movement whose dating is unknown. In fact, it can be observed in aerial photographs taken in 1956. The straightness of the upper scar and the fact that the soil near the crack does not indicate any surface tilt suggests a translational type of slide.

7.1.1 *Geological setting and location of the sliding plane*

The northern slope of the Sierra de Blancafort consists of sedimentary rocks whose age range from late Cretaceous to Paleocene. Six groups of materials have been distinguished in the bedrock. These units have been defined by means of surface observation and the statigraphic columns provided by the borings performed (Corominas and Moya, 2009). Figure 31 shows a representative geological profile.

The identification of the sliding plane was based on several indicators:

- Presence of frequent shearing planes. These planes can be easily identified due to the existence of slickensides or polished surfaces.
- Continuity and parallelism of shearing planes with respect to the potential sliding plane.
- Degree of core recovery and quality of the samples above the shearing plane. In case of sliding, the movement can break the most rigid strata and produce a massif of less quality when compared with non-slid rock samples.
- Degree of core recovery and quality of the samples above the shearing plane. In case of sliding, the movement can break the most rigid strata and produce a massif of less quality when compared with non-slid rock samples.
- The interpolation of the sliding plane among borings cannot be inconsistent.
- Finally, the correlation with inclinometric records.

7.2 *Laboratory tests*

This section summarises the results obtained in the laboratory on samples of natural and remoulded material taken from the Garumn clayey stratum where the failure surface is located. The natural conditions of the samples indicate densities ranging from 1.71 to 1.88 T/m^3, which correspond to void ratios of 0.96–0.68. Water content in most cases is close to 20%. Garumn clay is quite plastic (w_L = 54–57%; PI = 26–31%) (CH). 40% of the particles are smaller than 2 μm.

Since the landslide is a reactivated slide, the residual strength was measured on remoulded samples in the ring shear equipment. The measured angle ($\varphi'_{res} = 12 - 13^{\circ}$) is consistent with the clay plasticity.

The permeability to water was also measured in two samples subjected to a vertical stress of 300 kPa. Measured values ranged between 4.2×10^{-10} and 4.9×10^{-11} m/s.

7.3 *Water level and piezometer measurements*

Figure 33 shows the evolution in time of the water level elevation in the reservoir from January 1987 until 2009. The crack in the left valley slope of the reservoir was observed at the end of summer 2006, after the water level elevation dropped to elevation 430 (absolute minimum value since 1987, according to available data). The average daily drawdown velocity, before failure, was around 0.25 m/day and the daily maximum velocities were higher than 1 m/day during the last drawdown period. Note that the 2006 drawdown followed a long period (1992–2004) in which the level was consistently located above elevation 480. This period probably allowed reaching water pressures corresponding to high water elevations, even in those strata with

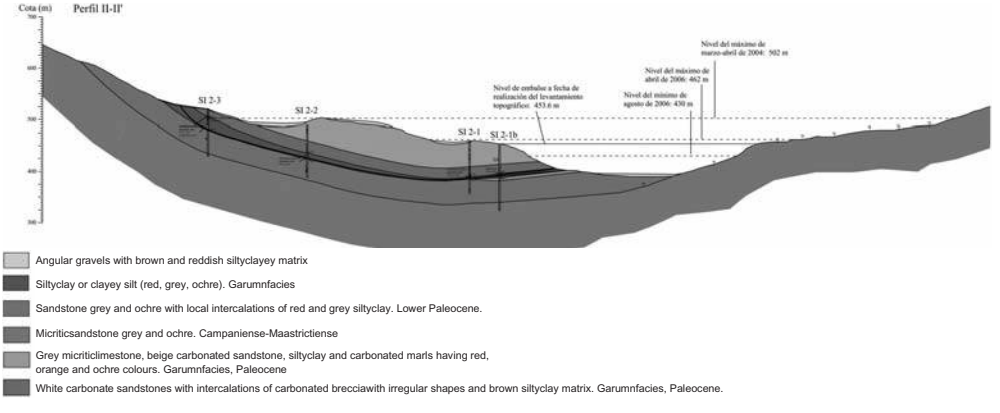


Figure 31. Geological Profile II (Corominas and Moya, 2009).

Figure 32. Striated and polished surface Garumn clay recovered from a boring sample.

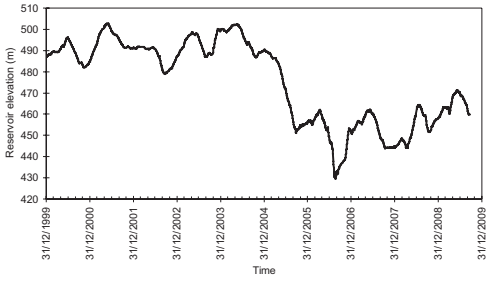


Figure 33. Time evolution of water reservoir elevation.

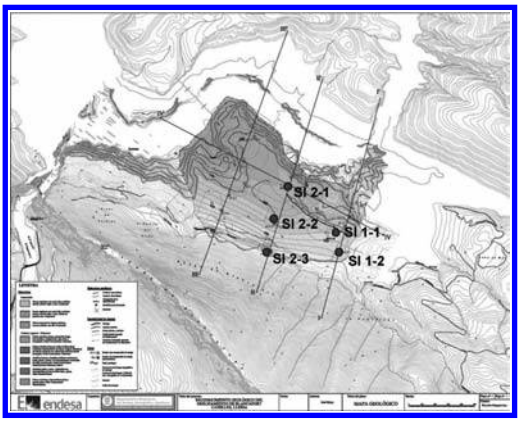


Figure 34. Position of piezometers installed after the failure.

lower permeability. This previous history should be taken into account when estimating the water pressure during drawdown.

Figure 34 indicates the location in plan view of the piezometers installed. Several sensors (three or four) were installed in each bore hole at different

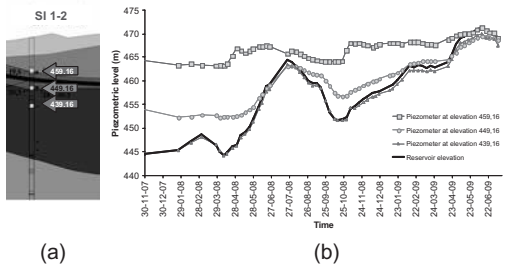


Figure 35. (a) Position of piezometers in borehole SI 1–2 (see Fig. 34); (b) piezometer records and reservoir elevation.

depths. Piezometers were located around the clayey layer where the failure surface was most likely to be found.

Consider the response of piezometrs in boring SI 1–2 in Figure 35. The recorded water pressures in the sandstone stratum follow the water level elevation in the reservoir. On the contrary, the piezometers located in the clayey stratum maintain high pressures, which in some cases exceed the pressures corresponding to the current water reservoir elevation. The value of the pressures recorded in SI 1–2 in the piezometer located within the clayey Garumn stratum remains constant and independent to the water level elevation. This result confirms the low permeability of the clay stratum, in contrast to the higher permeability of the sandstone and marls located below. The pressure in the upper part of the slide does not seem to be directly connected to the reservoir. High pressures were measured in the rock layer. These values of water pressure probably are a consequence of direct infiltration from rainfall or perhaps the results of other sources of water no well defined, linked to the geological structure of the area.

As a summary, the examination of piezometer measurements leads to the following conclusions:

– The hydraulic behaviour of the Garumn clay stratum seems to be independent from the lower sandstone stratum.
– The pressures in the sandstone stratum, except for the upper part of the slide, follows immediately the water level elevation in the reservoir. This behaviour is an indication of the high permeability of the sandstone.
– Pressures remain essentially constant in the clay and independent from water level elevation mainly during the first seven months of measurements when reservoir elevation was lower than 460 m. This is an indication of the difficulty to dissipate or increase water pressures within clayey layers where the sliding surface is located. It also points out the low “in situ” permeability of this clay level.

7.4 *Pore water pressure calculation*

The characterisation of the slide, the laboratory tests performed and piezometric measurements seem to indicate that the cause of the sliding was the high water pressures that remained within the low permeability clayey stratum, together with the absence of the stabilising effect of reservoir water due to the drawdown.

The analysis presented here was carried out by means of the finite element code Code_Bright, which can solve coupled flow/deformation problems in saturated/unsaturated media.

The calculation model is shown in Figure 36. The figure also shows a linear quadrilateral element mesh. Nodes have three degrees of freedom (water pressure and vertical and horizontal displacement). The mesh had to be refined mainly in the thinner clay stratum to ensure the correct calculation of water flow through materials in direct contact having values of permeability widely different among them. In order to simplify the model, the sequence of detailed stratification above the Garum clay level has not been specified in detail. The mobilised rock above the clay mainly consists of sandstone levels.

The stress-strain behaviour of the materials was characterised by means of a linear elastic law defined by Young's modulus and Poisson's ratio. Elasto-plastic considerations have limited effect when estimating drawdown-induced pore water pressure. This result is explained because the drawdown mainly results in an elastic stress path. In addition, the involved materials are highly overconsolidated rocks. Thus the elasticity hypothesis seems sufficient in this case. Table 4 shows the elastic values chosen, as well as the saturated permeability. Clay parameters were derived from laboratory tests, while rock parameters were estimated according to typical values due of lack of precise data.

The analysis requires the characterisation of materials under unsaturated conditions. The retention curves introduced in the calculations use the Van Genuchten model having the parameters indicated in Table 4 and plotted in Figure 37. Relative permeability is defined according to the cubic law ($k = k_{sat}k_{rel} = k_{sat}S_r^3$ where k_{sat} is the saturated permeability and S_r is the degree of saturation) for both materials.

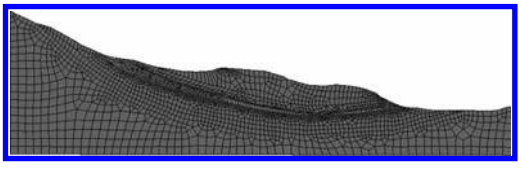


Figure 36. Finite element calculation model.

Table 4. Parameters for coupled hydro-mechanical calculations.

Parameter and unit	Clay	Rock
Young modulus (MPa)	500	2500
Poisson's coefficient	0.3	0.3
Saturated permeability (m/s)	$4.9 \cdot 10^{-11}$	10^{-5}
Van Genuchten parameters:		
P_0	0.3	0.03
λ	0.33	0.33
$S_{r\max}$	1	1
$S_{r\max}$	0	0

The known history of water level elevation, was modelled in the period October, 2002 until July, 2009. This allows establishing the pressures acting on the failure surface with the purpose of analyzing the stability at any time. In addition, the measurements taken in the piezometers installed in November 2007 could be compared with calculations in an effort to validate the numerical model. The initial pore water pressure assumed in the calculation is a horizontal hydrostatic profile.

The effect of rainfall has been incorporated in the simulation in order to model the water inflow through the upper part of the slope, which is approximately equal to the average value of the rainfall recorded in the region (400 l/m^2/year).

Figure 38 shows the water pressure contours calculated in August 21st, 2006, when water elevation of the reservoir reached elevation 431 m (the minimum values reached in the period 1986–August 2006). The effect of the low permeability of the clay layer and its continuity can be observed by the abrupt change of the contours.

The calculated pore water pressure distributions have been introduced in the calculation of the safety factor, by means of a limit equilibrium procedure to analyse the stability in the most critical situations previous to the failure.

However, before the analysis the failure is presented, the calculated pore water pressure will be compared with piezometer measurement recorded after the failure in order to check the reliability of results. Figure 39 shows such a comparison for the piezometers installed in Profile II.

Pore water pressure measured within the lower sandstone in SI 2–2, which follow precisely the reservoir elevation evolution, are well captured by the calculation. This is a consequence of a correct choice for sandstone permeability. Pressure measurements within the impervious clayey layer in SI 2–2, which are especially important for the subsequent calculation of safety factor, have been simulated quite satisfactorily. Pressures are lightly overestimated when the reservoir reaches low levels.

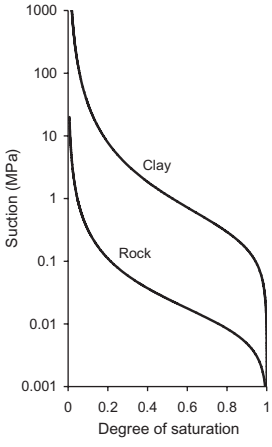


Figure 37. Retention curves used in calculation.

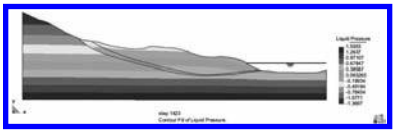


Figure 38. Calculated pore water pressure distribution on August 21, 2006 when reservoir elevation was at 431 m (see Fig. 33).

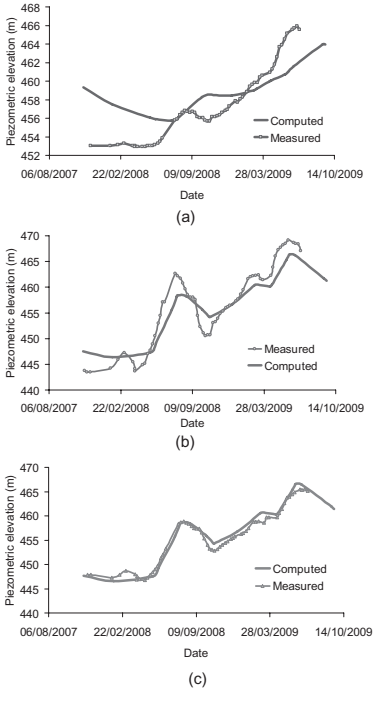


Figure 39. Comparison of calculated and measured pore water pressure in piezometers located in boring SI 2–2 at elevation (a) 429.88 m; (b) 419.88 m; (c) 409.88 m.

The major discrepancy appears in piezometers installed in SI 2–3. Reservoir level was always below the elevation of piezometers located in SI 1–3. In addition, the high sandstone permeability allows a rapid flow of rainfall infiltration towards the reservoir level. As a result, no positive pore pressures above the piezometer positions are calculated. Field measurements show a totally different response. In fact, water columns in excess of 15–25 m over the piezometers location are measured. This is probably a consequence of the external infiltration and a different local geological structure of pervious/impervious materials at the head of the slide. In subsequent stability calculations, this field information will also be considered.

7.5 *Safety factor*

The failure surface seems to be located entirely in the Garumn clay stratum. Therefore, the analysis of limit equilibrium only requires the definition of the soil densities above the failure surface, the strength properties of clay and the pore water pressure acting on the failure surface. The unit weight of the mobilized mass has been estimated equal to 21 kN/m^3.

The strength available on the failure surface was defined by a Mohr-Coulomb law. Since the slide is the reactivation of an ancient slide, the residual angle and a null effective cohesion should be used. The ring shear tests carried out in the laboratory indicate a residual angle of 12º under vertical effective stresses ranging from 100 to 250 kPa. Nevertheless, the vertical effective stresses on the failure surface range from 100 to 1500 kPa (some 800 kPa in average). Generally, the friction angle decreases with effective normal stress. On the other hand, during the geological history of the slide the failure surface probably underwent larger than current stresses. Later, once unloaded until reaching the current initial situation, the remaining friction angle would correspond to the lowest value in the past. This hypothesis would lead to a friction angle lower than the value measured in the laboratory (12º) and most likely close to 10º (Stark and Eid, 1994; Alonso, 2005). Both values will be considered in calculation.

The water pressures obtained by these calculations were introduced in the calculation of safety factor by limit equilibrium with the program GeoSlope. The model used is shown in Figure 40. The failure surface has been predefined following the geological interpretation. It is entirely located within the clay layer. Pore water pressure on the failure surface is imposed following the results obtained previously in the coupled analysis. The increase in strength due to suction was neglected in the unsaturated area.

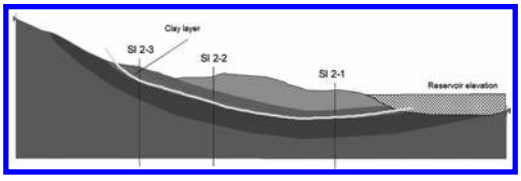


Figure 40. Calculation section for limit equilibrium analysis and pre-established failure surface in yellow.

Table 5. Safety factors calculated by limit equilibrium analysis.

	September 26th, 2005. Reservoir elevation: 451 m	August 21st, 2006. Reservoir elevation: 431 m
$\phi' = 12^o$	1.25	1.31
$\phi' = 10^o$	1.04	1.09

The two critical drawdown events (September, 2005 and August, 2006) have been analysed. The safety factors obtained are indicated in Table 5 for two clay friction angles, 10º and 12º. The values obtained for $\phi' = 10^o$ are very close to 1. Both drawdown situations were quite critical. However, the calculated safety factors are higher than 1 in both cases. Several factors can explain this discrepancy. It is uncertain if the analysed section is the most critical one. On the other hand, piezometers measurements indicate higher pore water pressure values in the upper part of the landslide. They were not introduced in the analysis reported in Table 5. If these measured values are introduced in the limit equilibrium analysis, maintaining the remaining values obtained in the calculation of pore water pressure, a safety factor equal to 1.01 is obtained for the situation in August.

7.6 *Rapid sliding*

An additional risk is the possibility of rapid sliding which will result in the impact of the sliding mass against the reservoir water and the subsequent generation of uncontrolled destructive waves. A clear reference is the Vaiont slide in northern Italy in 1963. Other cases have been described in the first part of the paper.

The phenomena that may lead to this rapid acceleration were discussed in Pinyol and Alonso (2009) and Pinyol and Alonso (2010). The coupled thermo-hydro-mechanical problem has been solved in a representative cross section of Canelles landslide consisting of two blocks. The failure surface was simplified by means of two planes: one under the upper block, which dips 18º, and another one under the lower block, next to the reservoir,

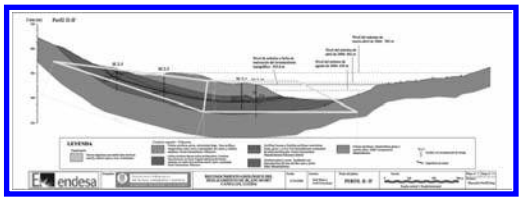


Figure 41. Calculation section considered for the thermo-hydro-mechanical analysis.

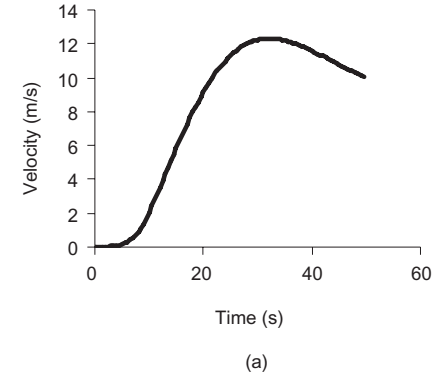
which is horizontal (Fig. 41). This geometry allows taking into consideration that as the slide progresses towards the reservoir, the deformed mass becomes, in principle, more stable. In fact, during sliding, the soil mass increases in the more stable lower block and decreases in the unstable upper block.

The analysis requires some thermo-hydro-mechanical properties of the clay stratum, where the failure surface is located. The values selected for water and solid particles correspond to well-accepted values found in handbooks of physico-chemical constants. The specific properties of the shearing band material (Garumn clay) were defined previously. The coefficient of compressibility was calculated through the Young modulus used previously (500 MPa) and $\nu = 0.3$.

It is difficult to estimate accurately the thickness of the shearing band, even if recovered samples are examined. In general, the thickness of the shearing band is related to the size of the soil particles. In this case, the failure is located in a clayey soil with $D_{50} = 0.01$ mm (aperture diameter of the mesh through which 50% of the material passes). For this value, a thickness of 5 mm is estimated for the shearing band.

The initial position of the slide considered for calculations corresponds to a water level elevation 470 m. For the dynamic analysis performed the water pressures on the failure surface are calculated under the simple hypothesis of hydrostatic distribution. The effect of water weight on stresses on the slope is also taken into account. Under these conditions, according to limit equilibrium calculations, Canelles slope was strictly stable and instability takes place whenever higher pressures develop in the clay stratum. The initiation of the movement (safety factors lightly lower than 1) was simulated by setting a pressure increase of 0.09 MPa throughout the failure surface of the lower block.

Figure 42a plots the evolution of velocity in time during a displacement of 200 m (Fig. 42b). According to calculations, the slide would accelerate, reaching 12 m/s in a few seconds, after travelling some 300 m inside the reservoir. At the beginning, the sliding velocity remains low, close to a few millimeters per second. During the first



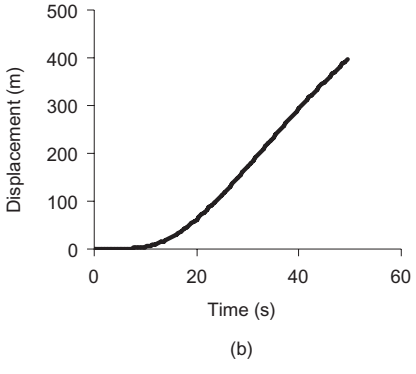
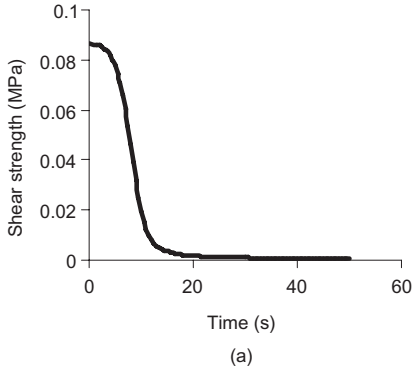


Figure 42. Calculated evolution of sliding velocity and displacement considering the thermo-hydro-mechanical effects.



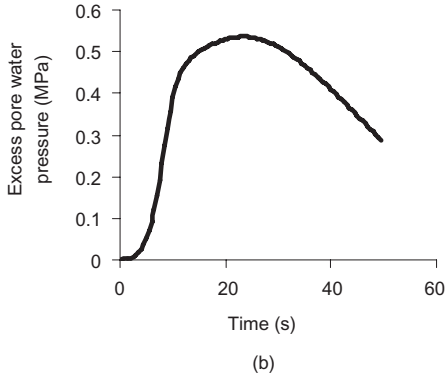


Figure 43. Evolution of (a) the frictional effective strength and (b) water pressure in the failure surface under the lower block.

seconds, the excess pore pressure accumulated in the shear band is not enough to significantly reduce the effective frictional strength.

Figure 43 shows the evolution of strength and the excess in water pressure during the same period of time. When pore pressure in the shear band increases (up to 0.5 MPa), shearing strength decreases and drops to values close to 0. This explains the high velocities reached by the slide.

If the effects of heat are neglected, the maximum velocity reached by the slide becomes significantly lower (12 cm/s). The velocity at $t = 40$ s is null, after travelling 3 m towards the dam.

8 EXCAVATIONS AND LANDSLIDES

Dam construction involves always excavations of varying importance in the foundation soils and rocks. They may lead to unstable soil or rock conditions.

In the case of excavations to build dam foundations the change in geometry is not permanent. But even in the transient period of excavation and dam construction stability conditions should be analyzed and ensured.

Consider in Figure 44a the excavation for a gravity dam in a sequence of layered siltstone and sandstone of the Weald period. This is a soft rock whose strength and degradation properties are controlled by the clay-rich layers as well as some mica-rich discontinuities. The sketch shows the excavation (about 12 m deep) and the dip of excavated strata. The steep slope became eventually unstable after a certain period of exposure to atmospheric conditions. Cuts in clay-dominated soils and rocks experience typically continuous decrease in safety conditions after excavation. This is a consequence of the dissipation of the negative effects pore pressures generated during the excavation unloading. In the case illustrated In Figure 44a an additional aspect was the significant weathering of the exposed Weald materials. The slide shown in Figure 44b was a planar motion following the dip of the sedimentation layers. In this case a rough estimate the available shear strength at the sedimentation planes was equal to the dip of the strata. These local failures do not have necessarily

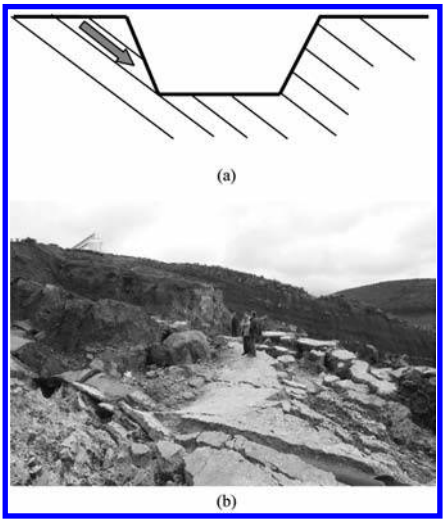


Figure 44. (a) Dam excavations in a layered rock and potential instability; (b) Failure in Weald claystones and sandstones.

a negative effect on dam stability but they may result in damage to auxiliary installations excess of excavation and time delays during construction.

Two dam structures require also excavations of significant importance: diversion tunnels and spillways. As an illustration of potential risks, a recent and serious case, presented in Figure 45, shows the large excavations necessary to build two large diversion tunnels (lower part of the photograph). The dam to be built is a 200 m high concrete face rockfill dam. The rock is a complex sequence of volcanic materials crossed by faults. The excavation shown in Figure 45 triggered a large block type slide ($2{,}5 \cdot 10^6$ m^3) limited by a number of faults. The unstable block was close to the upstream dam face and one of the priorities of the dam construction was to stabilize the slide. Figure 46 shows some calculations performed (in a 2D cross section) to examine the effect of one of the stabilization measures designed: The construction of a large concrete block at the tunnel entrance, restoring in part the excavated rock weight. The upper part of the slide was also unloaded. The tunnels in this case cross some of the faults involved in the slide and the may be used also as reinforcing keys. The stabilized slide should exhibit sufficient safety margin against the submergence of part of the large block but also against rapid drawdown, a subject examined also in this paper.

Two final comments in connection with this case are given. Firstly, the extreme significance of taking care of instabilities to ensure the success of the entire project. In fact, if no acceptable stability conditions are achieved, the dam could not probably

Figure 45. Excavations to build diversion tunnels (lower part of the photograph).

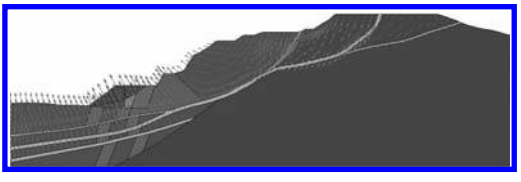


Figure 46. Calculated displacements as a result of rapid drawdown. The figure shows the also part of the complex system of faults.

be built in the intended location. The second one is the very large effort required to ensure the stability conditions of a very large slide in a complex geological setting. The case outlined involved a truly three-dimensional motion bounded by non regular warped faults. The field work necessary to identify the slide geometry and the relevant geotechnical properties is outstanding. In addition, field monitoring was very comprehensive and required the continuous attention of an entire technical team. Interpreting field data and developing the relevant calculation models, robust enough to generate confidence, is also a major task at the limits of present geotechnical and geomechanical knowledge.

9 IMPLICATIONS FOR DAM SAFETY

The discussion given in the paper and the number of case histories described suggest a few additional considerations and preliminary conclusions:

- Fast sliding is not an isolated or very rare event. It is not associated to very large landslides (as the well known Vaiont case may suggest). Cases in which landslides reach dangerous velocities when they hit the reservoir are being progressively reported.
- Static analysis and "standard" dynamic analysis are unable to predict the development of fast

landsliding. In fact, even very basic geometrical considerations suggest than once a motion is triggered it should reach a static equilibrium after a reasonable displacement. The accelerated motion is explained if mechanism reducing the available strength on the sliding surface can be rationally proposed. The following mechanisms were listed: strain softening, liquefaction of loose granular materials, crushing of particles on the shear band and thermal interactions. Strain softening is possible in brittle materials and first time failures. It remaining to be demonstrated that it may lead to high velocities. Thermally induced pore pressure may justify a complete destruction of available strength and it explains the Vaiont case.

– Recent contributions on the analysis of fast landslides of a displacement type (liquefaction and flows are excluded) indicate that a key parameter to decide the possibility of a fast landslide is the permeability of the soils/rocks in the vicinity of the sliding surface. "Impervious" materials having $k < 10^{-9}$ m/s lead to acceleration motion. "Pervious" materials, $k > 10^{-8}$ m/s, are safe in this context. However, the exact reasons for the triggering of an accelerated motion are not known. Landslides may exhibit a long history of slow motions without accelerating. But the proceeding criteria based on permeability is a promising start which should be validated against new case histories.
– Two triggering mechanisms are typically associated with reservoirs: the submerged of the slope toe and the rapid drawdown. The fist one is well understood and well captured by current methods for analyzing stability. In general, there exists a critical level of the reservoir which leads to a minimum static safety factor. This critical elevation depends on the geometry of the sliding surface. The critical elevation of water over the position of the existing sliding surfaces ranges typically between 0.3 and 0.5 of the landslide vertical dimension.
– A proper analysis of rapid drawdown effects requires a joint consideration of flow and stress-strain behaviour of the materials. Simplified procedures described in textbooks and papers (flow method, undrained methods) are likely to procedure errors. However, it is not a simple task to decide that a simplified method would be a safe or a conservative method. This comment has been illustrated in one of the case studies described. The conclusion is that the analysis should be rigorous.
– When reservoirs reach high water level, rapid drawdown are difficult in practice because of the large flow rates required (think in the relationship between reservoir volume versus water level). In addition, a full submergence of a potential slide has naturally a significant safety factor for reasons given above. Therefore the significance and risk of drawdown tends to decrease when the reservoir level is at some intermediate or low position. The positive side of this scenario is that a drawdown induced sliding will find a safeguard difference of elevation between dam crest and reservoir level. Of course, there are approximate comments inspired by some general qualitative reasoning and the case study of Canelles dam. However one may find alternative situations and every case requires a specific analysis.

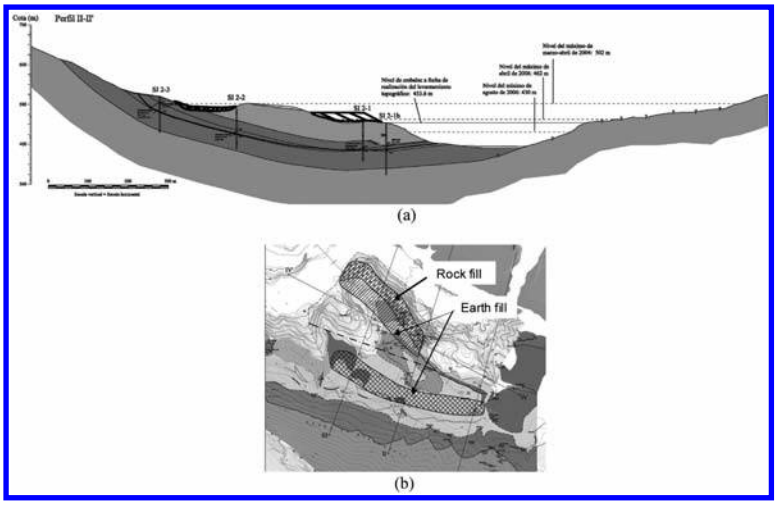


Figure 47. Proposal of stabilization by weight transfer: excavation and fill zones: (a) Cross section II; (b) plan view.

– It has also been an experience associated with Canelles case that a "normal" management of the reservoir can hardly guarantee a situation of no risk just by controlling the rate of drawdown. Therefore, if a risk of fast landslide is appreciated the best solution is to stabilize the slide. In case of large landslides (many examples in this report go beyond 20 Mm^3) there are no much alternative and a weight transfer procedure is probably one of the most practical ones to increase safety. In the case of Canelles the weight transfer stabilization was analyzed and the proposal is synthesized in Figure 47.
– Excavation-induced instability has also been discussed with the help of two cases. One of them brings the issue of the continuous reduction of stability in clay or silt rich materials because of pore water dissipation and weathering effects. The second case illustrates the decisive risks for dam construction posed by large induced slides and the substantial effort required to analyze and stabilize major landslides.

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La realización de la presa de Koudiat-Acerdoune (Argelia): Un desafío geológico y técnico

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RESUMEN: La presa de Koudiat-Acerdoune se ubica en las sierras del Atlas Tellien, región estructurada geológicamente por cabalgamientos activos, iniciados durante la orogenesis alpina, generadores de actividad sismica, afectando formaciones calcáreas del Jurasico, y margas-calcáreas del Cretaceo. La presa, una estructura gravedad en HCR, tiene unos 120 m de altura, fundada sobre esquistos margosos. Durante los trabajos de excavaciones ocurrieron una serie de inestabilidades en el estribo izquierdo hasta constar que un deslizamiento mayor estaba a punto de poner en movimiento unos 2,5 millones de metros cúbicos de rocas, con una línea de ruptura pasando por debajo del nivel de proyecto para la fundación de la presa. Una campaña complementaria de investigaciones geomecánicas fue entonces implementada, resultando en una profunda revisión de las características de la fundación de la presa. El diseño de la presa fue cuidadosamente revisado para tener en cuenta la estabilidad estática y sísmica sobre una fundación de calidad menor que esperada. El objeto de la conferencia es presentar los elementos principales de este proyecto y los conceptos inovativos que tuvieron que ser movilizados para enfrentar estas condiciones geológicas imprevistas particularmente complejas, y los primeros retornos de experiencia traídos por la puesta en servicio de la obra en 2009.

1 CONTEXTO

1.1 *Introducción*

Ubicada sobre el Oued Isser en las serranías norte de Argelia, a unos 90 km al sur-este de Argel, la presa de Koudiat-Acerdoune (Figura 1) embalsa un reservorio de capacidad de unos 640 hm^3 destinado a segurizar recursos en agua para abastecer las necesidades de la región de Argel y las "wilayas" (distritos o comunidades) de Bouira, Medea, M'sila and Tizi Ouzou. Situado a unos 90 km al sur-este de Argel, es un elemento clave del amplio programa de infraestructuras hidráulicas desarrollado por la Agence Nationale des Barrages et Transferts, con la asistencia de Coyne et Bellier Ingénieurs-Conseils.

La geología de la región del Atlas Tellien donde se ubica el proyecto es esencialmente estructurada por sistemas de cabalgamientos Miocenos desplazados hacia el sur durante la orogenesis Alpina; estas capas de cabalgamiento afectan formaciones de calcáreo Jurasicos, y de calcáreos

Figura 1a. Vista global del sitio de presa y de embalse.

Figura 1b. El cierre del sitio de presa.

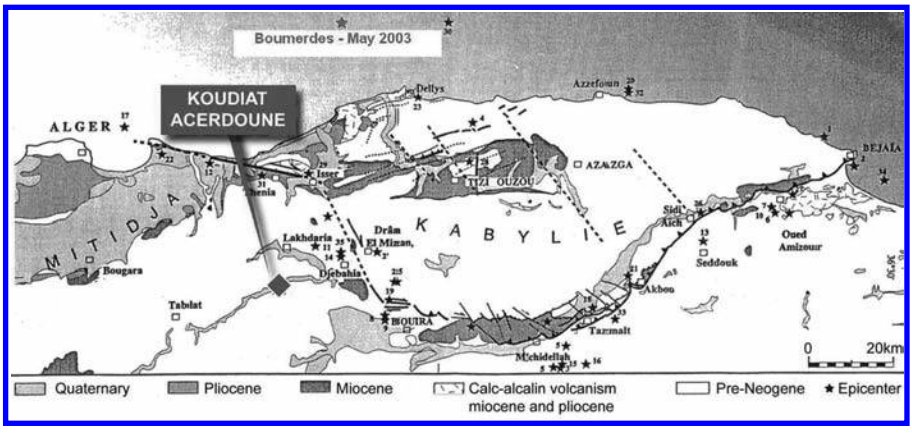

Figura 2. Mapa sismotectonico de Kabylia y region este de Argel (modificado de Boudiaf, 1996).

margosos Cretáceos. La sismicidad en esta región se encuentra asociada a fallas transversas, resultando en seísmos relativamente frecuentes que, a pesar de intensidades bajas a moderadas, pueden producirse a corta distancia del sitio. Eventos sísmicos de diseño han sido evaluados por estudios regionales de sismicidad histórica (Figura 2).

La presa de 120 m de altura es una estructura gravedad de HCR soportando un aliviadero libre de capacidad 7000 m^3/s. La fundación consiste en esquistos margosos y calcáreos margosos desformados por la tectónica.

1.2 *Diseño sísmico inicial*

El estudio de riesgo sísmico determinó los eventos de diseño siguientes:

- Sismo Básico de Operación: magnitud 5,6 a 6 km, aceleración basal de pico 0,20 g;
- Sismo Maximo de Diseno: magnitud 6,1 a 6 km, acceleracio basal de pico 0,30 g.

La respuesta dinámica de la presa había sido estudiada por análisis modal en modelos FEM bidimensionales, seguidos por recomposición espectral. Las aceleraciones espectrales aplicadas a la presa, en conjunto con las presiones hidrodinámicas (método de Westergaard) habían determinado los esfuerzos dinámicos en el cuerpo de presa y al contacto con la fundación. Una atención particular había sido dirigida hacia la eventualidad de esfuerzos internos en tracción en la cara aguas arriba, hacia el riesgo de fisuración iniciándose en esta cara aguas arriba, y también hacia las tensiones ejercidas sobre el contacto con la fundación.

El objetivo del estudio de diseño original dirigido hacia la optimización de la sección típica del cuerpo de presa, había sido de chequear que:

- bajo el SBO, no hay riesgo de fisuración que podría afectar significativamente la distribución de sub-presiones dentro del cuerpo de presa o al interfaz con la fundación;

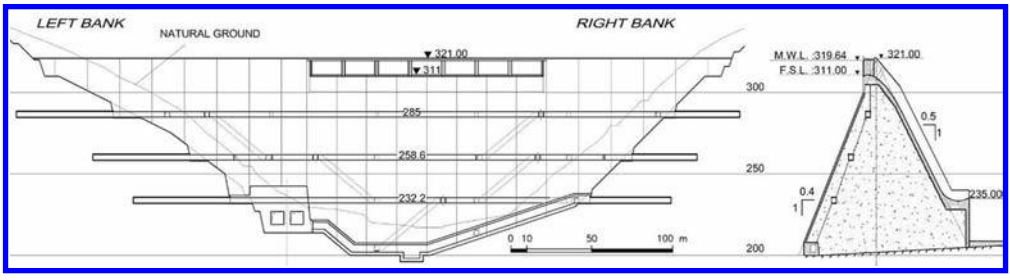

Figura 3. Proyecto inicial—Cortes longitudinal y transversal.

– bajo el SMD, la estabilidad es asegurada en la configuración post sismo, asumiendo que la plena presión se propaga en las zonas potencialmente fisuradas en el cuerpo de presa.

La disposición principal había sido de escoger una cara aguas arriba inclinada a 0,4 H/1V, requerida para asegurar la estabilidad en deslizamiento sobre la fundación; en consecuencia, el cuerpo de presa en el diseño inicial ya era bastante masivo con un ratio global de 0,9 H/1V (Figura 3). El riesgo de fisuración era limitado con el uso de HCR con alto contento en mortero y la provisión de una cara aguas arriba en hormigón convencional.

2 DIFICULTADES GEOLOGICAS

2.1 *Primeros movimientos e investigaciones adicionales*

Al inicio de las obras, el programa del proyecto incluía excavaciones significativas a lo largo del pie del estribo izquierdo entero, requeridas para la construcción de las obras de derivación constituidas por un canal de hormigón armado de unos 300 m de largo.

Durante estas excavaciones (a partir del fin del 2002), una serie de eventos de inestabilidades en las laderas revelaron que los esquistos margosos y calcáreos margosos altamente plegados y desformados en este estribo izquierdo, que también se meteorizaban rápidamente una vez expuestos a la intemperie, no estaban comportándose tal que asumido en el proyecto inicial, y también que la situación natural de las laderas en el sitio presentaba muy bajas márgenes de estabilidad (ref 2 y 3).

Esta sucesión de inestabilidades de laderas se intensificó gradualmente, requiriendo numerosos ajustes en los perfiles de excavaciones, reparaciones y refuerzos, conjuntamente con los ajustes que correspondían en las obras de esta margen izquierda del proyecto, donde la construcción del canal de desvío era la primera prioridad.

En mismo tiempo, se decidió revisar cuidadosamente la configuración de las excavaciones a ejecutar en el estribo derecho, las cuales habían sido retardadas por las dificultades encontradas en la margen izquierda, y proveer los refuerzos requeridos para estabilizar este estribo izquierdo y protegerlo contra la exposición directa a la intemperie. Esta operación necesito obras adicionales significativas, comparado con el proyecto inicial en este estribo derecho:

– unos 450,000 m^3 de excavaciones adicionales, requeridos para suavizar las pendientes excavadas;
– unos 41,000 m de anclajes pasivos profundos, de 24 a 30 m en longitud, asociados con drenaje denso, requeridos para proveer un factor de seguridad suficiente en la zona de las obras en configuración definitiva;
– unos 35,000 m^2 en hormigón lanzado, reforzado con malla soldada, para proteger el entorno de las obras de la exposición directa a la intemperie.

Estos trabajos adicionales en el estribo derecho lograron su meta (Figuras 4 y 5).

Durante este periodo, como estas dificultades geológicas ponían en cuestión la factibilidad de una tal presa en este sitio, un programa significativo de investigaciones adicionales fue elaborado y ejecutado para proveer datos nuevos para una revisión profunda de las características de la roca de fundación, y proveer los parámetros relevantes para una revisión completa del diseño de la presa. Como las metas principales de este programa eran la evaluación de la resistencia al corte y la deformabilidad de la roca de fundación, el programa de investigaciones adicionales incorporó:

– unos 15 sondeos con diámetro suficiente y procedimientos cuidadosos de recuperación, para proveer muestras adecuadas para ensayos en laboratorio de mecánica de rocas (no es siempre fácil de recuperar muestras intactas en este tipo de rocas margosas densamente fracturadas, plegadas, y desformadas), con objeto la determinación de parámetros y características de resistencia al corte de la roca misma y de los materiales llenando las fallas;

Figura 4. Estribo derecho—Excavaciones, refuerzos y protecciones a gran escala.

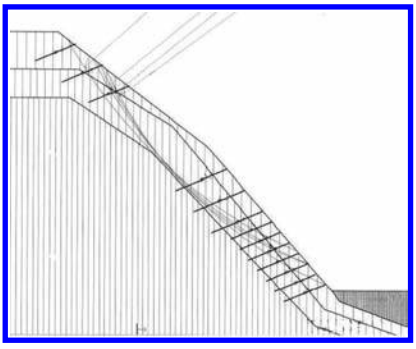

Figura 5. Computaciones de anclajes pasivos en el estribo derecho.

Figura 6. Estribo izquierdo—Reconfiguración de la parte superior—Noviembre 2004.

– 2700 m de perfiles de sísmica refracción asociados con 3 dispositivos de medición de velocidades sísmicas en cross-hole;
– 8 ensayos de deformación a la placa en galerías (con gatos hidráulicos), en ambos estribos (Figura 7).

2.2 *El deslizamiento mayor y sus consecuencias*

Al inicio del verano de 2004, apareció que a pesar de todos los refuerzos y reparaciones realizados y del cuido general puesto en estos trabajos, un derrumbe mayor afectando todo el estribo izquierdo del proyecto de presa, involucrando un volumen de rocas estimado a unos 2.5 millones de metros cúbicos, estaba preparándose lentamente en la margen izquierda. Este derrumbe tenía unos 350 m de largo, 150 m en altura, y cortaba los esquistos margosos unos 20 m por debajo del nivel de fundación (Figura 7). La apertura progresiva de fracturas y grietas en la superficie del estribo, la ruptura del revestimiento de hormigón en galerías de drenaje ya realizadas en profundidad del estribo, y el análisis de registros de controles topográficos realizados para la seguridad de las obras, no dejaban duda sobre la realidad y la extensión de los movimientos.

Dada la importancia estratégica del proyecto para el abastecimiento en agua de toda la región de Argel, las autoridades decidieron rápidamente la implementación de un plan de emergencia para parar los movimientos y proteger el estribo izquierdo antes de la llegada de la próxima estación de lluvias. El plan consistía básicamente en:

– la excavación de la mayor parte superior del estribo, para aliviar una parte significativa del peso motor del movimiento, y con pendientes suavizados para mejorar el margen de seguridad;
– el refuerzo y la protección cuidadosa de la superficie excavada, para proveer el perfil final de esta parte superior.

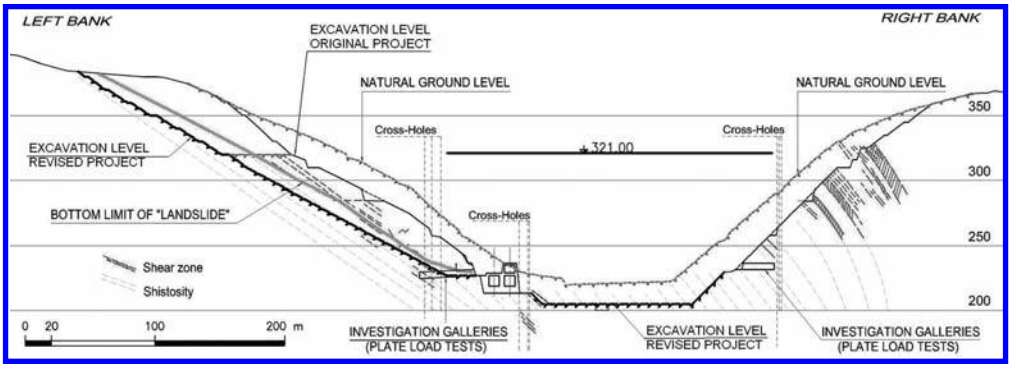

Figura 7. Sección transversal detallando las investigaciones adicionales, la base del deslizamiento en el estribo izquierdo, y el nivel revisado para la fundación de la presa.

En los trabajos de primera urgencia se removieron unos 1 000 000 m^3 de excavación adicional, y disposiciones de seguridad significativas fueron tomadas para medir en continuo, documentar y registrar los movimientos en superficie en el area de las obras, y para proveer un sistema de advertencia para la seguridad de los trabajadores operando al pie del estribo izquierdo.

Estos trabajos de primera emergencia en la margen izquierda, diseñados, implementados y ejecutados rápidamente durante el otoño 2004, lograron su meta (Figura 6). Los movimientos en el estribo izquierdo deceleraron y pararon, aliviando la urgencia de la situación y haciendo posible la finalización de las obras de derivación, que por suerte no habían sido afectados por los movimientos, y la terminación del programa de investigaciones adicionales, lo todo en condiciones razonables.

3 LA RECONFIGURACION DEL PROYECTO

3.1 *Revisión de los parámetros geomecánicos de la fundación rocosa*

Las investigaciones adicionales realizadas en 2003–2004 proveeron los parámetros requeridos para revisar el modelo geomecánico de la fundación rocosa. Comparado con el diseño original, los resultados fueron:

- una reducción significativa en la resistencia al corte estimada en los esquistos margosos (típicamente encontrados en 0,15 MPa en cohesión y 29° de fricción interna, en lugar de 0,5º MPa y 35°);
- una estimación del modulo estático de rigidez (primera carga) en la fundación rocosa, variando típicamente entre 1500 MPa a nivel de fundación, a unos 3000 MPa unos 30 m debajo del nivel de fundación. El modulo dinámico fue estimado variar entre 3000 y 6000 MPa en los mismos niveles;
- un ajuste de la profundidad de excavación requerida para encontrar una fundación adecuada para el diseño revisado, que consistió en profundizar de 30 m y extender de 60 m dentro del estribo izquierdo (Figura 7).

3.2 *Redimensionamiento de la presa y recalificación del HCR*

La natura de las rocas de fundación revelada por los trabajos de excavación no permitía proseguir con la construcción de una presa convencional de gravedad (aun en HCR), a causa de la baja resistencia al corte de la fundación, combinada con su alta deformabilidad.

Soluciones alternativas que hubiesen sido posibles en un tal sitio, como presa de escollera o de tierra, fueron rápidamente descartadas por el contexto particular del proyecto ya en marcha, con trabajos de excavación de fundación y varias obras anexas (derivación, vaciado de fondo) ya parcialmente realizados. Un cambio completo del tipo de presa hubiera conllevado atrasos y sobrecostos excesivos, y además de las limitaciones ya expuestas, una presa convencional en materiales sueltos hubiese inducido otros problemas difíciles de solucionar, particularmente en cuanto a desplazar sobre uno de los estribos el aliviadero de 7000 m^3/s.

Una alternativa que hubiese sido adaptada a las condiciones del sitio fue considerada: la “Presa Simétrica de Terraplén Endurecido, con cara impermeable” (Faced Symmetrical Hardfill Dam), un concepto relativamente innovador que había sido construido ya en algunas ocasiones (presas de pequeño a medio tamaño). Este tipo de presa es apropiado para una fundación tan problemática, pero hubiera traído modificaciones costosas y

complicaciones constructivas, en particular con la necesidad de construir la cara impermeable aguas arriba después del cuerpo de presa.

Fue entonces necesario de encontrar un concepto de presa gravedad HCR ubicado entre la presa gravedad convencional y la presa simétrica de hardfill, que podía integrar las características particulares de la fundación y los materiales disponibles para la elaboración del HCR, y asegurar un buen comportamiento estructural bajo condiciones de cargas normales y excepcionales, especialmente bajo solicitación sísmica. En este sentido, la primera necesidad era la definición de una nueva sección típica de la presa, y la revisión de las características de los materiales a partir de los cuales estaba para construir (HCR), así que eventuales arreglos específicos (modo de colocación del HCR, impermeabilidad de la presa, etc.).

El nuevo perfil optimizado tenía que asegurar la estabilidad de la presa en materia de deslizamiento sobre su fundación, tomando en cuenta el comportamiento mecánico de ésta. Dada la forma del perfil de la presa, las características de la fundación, y el nivel significativo de solicitación sísmica, el nuevo diseño tuvo que ser validado por medio de:

- análisis de distribuciones de tensiones y deformaciones en el cuerpo de presa y en la fundación, usando modelos de elementos finitos bidimensionales;
- integración de la alta deformabilidad y heterogeneidad de la fundación, considerando eventuales variaciones del módulo en profundidad así que de una margen hacia la otra;
- análisis del comportamiento dinámico del conjunto presa/fundación y su respuesta a diferentes espectros sísmicos;
- análisis de posibles excursiones fuera del comportamiento reversible bajo solicitaciones sísmicas, con computación de deformaciones irreversibles, usando el método de Newmark por pasos de tiempo, con los registros temporales de aceleraciones del sismo de referencia (sismo de Boumerdès)

Las principales modificaciones resultando en la presa fueron (Figura 8):

– selección de un perfil mas simétrico y mas espeso, con pendientes aguas arriba y aguas abajo de 0,65 H/1V a fines de limitar el impacto sobre las obras ya construidas (derivación, excavaciones en el estribo derecho, cámara de válvulas del vaciador de fondo);
– rediseño completo de las excavaciones en el estribo izquierdo y le cauce del río, y en cierto extenso en la base del estribo derecho;
– reducción a 11 MPa de la exigencia de resistencia del HCR determinada en muestras intactas (comparado a la exigencia de 15 MPa del proyecto original). Esto en coherencia con la calidad baja de los materiales disponibles para elaboración de áridos, en los cuales se incorporó un complemento de finos calcáreos de unos 110 kg/m^3 para reemplazar los finos plásticos naturales inapropiados, y con una cantidad razonable de cemento de unos 120 para limitar el riesgo de calentamiento al fragüe y las consecuencias en términos de contracción de enfriamiento y riesgo de fisuración térmica;
– cuido especial de la cara aguas arriba, de hormigón convencional vibrado, levemente reforzado horizontalmente para limitar el riesgo de fisuración.

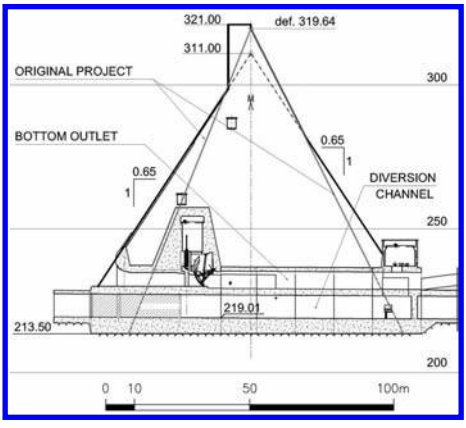

Figura 8. Perfil revisado de la presa.

3.3 *Reconfiguración del aliviadero*

Las obras del proyecto original incluían un aliviadero libre diseñado para los 7000 m^3/s de la crecida de proyecto, con solamente un salto de esquí (slip bucket) para la restitución al río. El caudal así restituido era previsto caer transversalmente al lecho del río, con un caudal especifico al impacto de unos 70 m^3/s/m.

Las dificultades geológicas que habían marcado los trabajos de fundación para la presa, también llamaron la atención sobre la sensibilidad de estos esquistos margosos frágiles a la erosión potencial en el lecho del río y al pie de ambos estribos, inmediatamente aguas debajo de la presa.

Un estudio de oficina confirmó:

– el riesgo de erosión profunda al pie de ambos estribos, particularmente el estribo derecho, con el diseño original;
– la necesidad de un nuevo diseño para proveer un mejor arreglo del impacto sobre el fondo del valle, evitando el riesgo de erosión y de

desestabilización de los estribos aguas debajo de la presa;
- dentro del análisis de soluciones posibles, la posibilidad de mantener un aliviadero libre, pero dividido en 7 canales separados, cada uno terminado por un salto de esquí específicamente diseñado para repartir el agua al impacto longitudinalmente en el valle, en lugar de distribuir el caudal transversalmente con salto de esquí de diseño mas básico;
- con esta disposición, la posibilidad de restituir en el valle chorros de agua paralelos repartidos en lo ancho del valle, con caudales unitarios al impacto bastante bajos debido a la distribución longitudinal, estos caudales unitarios siendo modulables en correspondencia con la ubicación del impacto (menor caudal unitario al impacto para los chorros impactando al pie de los estribos, mayor caudal unitario al impacto para los chorros impactando en el eje del valle, donde algo de erosión es menos problemático).

Este nuevo diseño fue objeto de ensayos cuidadosos en modelo físico a fines de 2005, en conjunto con el diseño original, en un modelo hidráulico a escala 1:100 con lecho móvil. Estos ensayos confirmaron la adecuación del nuevo diseño para controlar los riesgos de erosión, comparado con el diseño original, y permitieron ciertas optimizaciones en la configuración del aliviadero (Figura 9). Este nuevo diseño ha sido construido (Figura 10).

4 COMPORTAMIENTO DE LA OBRA EN SERVICIO- DISPOSICIONES PARTICULARES DE VIGILANCIA

La presa ha sido puesta en servicio en Julio de 2009 con el cierre del canal de desvío. Dada la capacidad del embalse comparado con los aportes promedio del río (el embalse es de 3 veces el volumen de aportes anuales), el llenado se realiza muy progresivamente.

La presa ha sido cuidadosamente instrumentada para analizar su comportamiento y poder advertir cualquier anomalía. El análisis de los movimientos de la presa con el llenado del embalse a media altura, deja inferir un modulo de rigidez

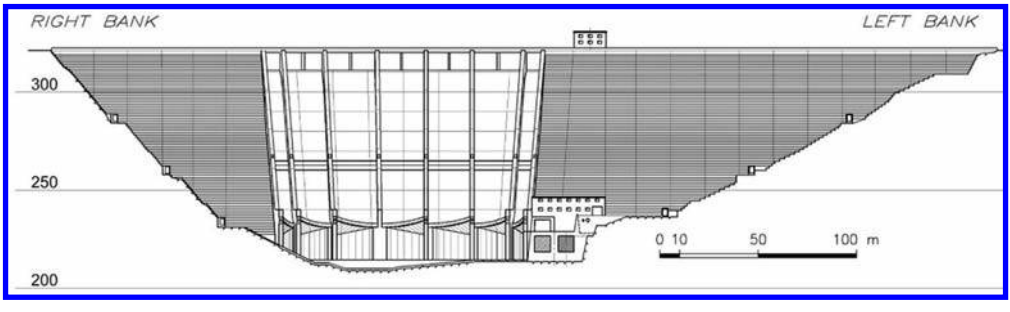

Figura 9. Vista lado aguas abajo del proyecto revisado.

Figura 10. Vista de la presa en fin de construcción del lado aguas abajo—Abril 2009.

promedio de la fundación de unos 6 000 MPa, significativamente encima de los valores esperados (se ejecutaron tratamientos densos de inyecciones de consolidación en esta fundación). Los caudales de perdidas colectados en la red de drenaje quedan el los valores razonables.

Dada la historia muy particular de la presa, se recomendó de continuar con una vigilancia específica hasta el llenado completo del embalse.

También tomando en cuenta la natura especial de estas formaciones de esquistos margosos se recomendó implementar una vigilancia específica de las laderas del embalse, particularmente al primer llenado, por medios topográficos.

5 CONCLUSIONES

Comparativamente al diseño original, los estudios realizados para reconfigurar y revisar el proyecto revelaron la factibilidad y el interés económico de un diseño innovador para una presa de HCR de gran tamaño en condiciones difíciles y un ambiente sísmico agresivo, en los cuales un tipo de presa de natura totalmente diferente podría normalmente haber sido escogido.

Este nuevo concepto cuidadosamente chequeado, fue aprobado por el dueño y permitió proseguir con la construcción, permitiendo también adaptar la exigencia de resistencia del HCR a la calidad real de los materiales disponibles en el sitio.

En base a los resultados logrados en estos estudios y en el comportamiento de la obra, será interesante continuar el desarrollo de estos conceptos y esta experiencia, para analizar hasta que extenso puede ser aplicable en condiciones previamente consideradas como poco compatible con presas gravedad en HCR.

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Mantenimiento de presas desde la optica de un monumento historico, cultural y turistico

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RESUMEN: España, así como algunos países Árabes y otros de la Mesopotamia [(del griego: región entre ríos), y (del conocimiento específicos entre los Ríos Tigres y Eufrates)], y de Babilonia, hace varios Siglos por medio de las primeras comunidades que abandonaron el nomadismo hicieron surgir los primeros núcleos urbanos en esas regiones, induciendo el desarrollo de un complejo sistema hidráulico que favorecía la utilización de los bañados, evitaba inundaciones y garantía el almacenamiento de agua para utilizarlas en las estaciones mas secas, haciendo necesaria la construcción de estructuras para mantener algún tipo de control sobre el régimen de los ríos. Solamente el trabajo colectivo permitió que se pudiesen dominar los ríos por medio de construcción de presas y acueductos. En la actualidad muchas de esas estructuras todavía mantienen su función original y otras se tornaron monumentos históricos-culturales-turísticos. Citar ese conjunto Histórico de Bienes Públicos puede hacer surgir una indagación: *¿Que tiene que ver la Historia y la Cultura con relación al Mantenimiento de Presas?*

En el texto siguiente, el Autor procura debatir esa indagación al considerar la necesidad de Inspección, el Cuidado (¡?) y las Acciones (¡?) de los Organismos Públicos responsables por guardar y mantener esas estructuras. El Autor tiene la certeza de que el tema es polémico, provoca contrariedades y, por eso mismo ve la razón de generar discusiones!

1 HISTORICO

La comunidad técnica dispone de varios libros[1] a [3] y publicaciones de seminarios, congresos y revistas, evidenciando la existencia de represas construidas hace varios siglos, que todavía demuestran y están garantiendo su utilidad original. En ese amplio acervo publicado hay sin embargo, presas-monumentos históricos que poseen un mantenimiento-conservación precario. Aquí cabe la primer pregunta:

☞ *¿Quien debe cuidar de esas presas-monumentos, o de eses Bienes Públicos?*

De una manera semejante, se puede afirmar (testimonio del propio Autor) que en varios Países hay presas, construídas contemporaniamente, en condiciones mas precarias, de que aquellas citadas en el acervo histórico y que todavia perpetuan su utilidad.

2 USO DE LOS RIOS

Al considerar el uso de los ríos, se pueden mencionar los siguientes:

- Transporte;
- Irrigación;
- Pesca e Caza;
- Abastecimiento:
- Minería;
- Energía;
- Fertilización aluvial;
- Recreación;
- Turismo;
- Contribución Climática

Queda en evidencia una multiplicidad de beneficios de que la Humanidad puede disfrutar, si cuida bien y si convive bien con los mismos. O sea si esa convivencia es racional en la distribución y abundancia de los seres vivos y de las interacciones que determinen su distribución.

3 LOS ASPECTOS LEGALES

Los diferentes Usos y Costumbres inducen a la creación de Normas-Códigos-Leyes, que buscan armonizar las ventajas y responsabilidades. Y así, de forma contemporánea se ve una creciente mejoría de las exigencias y cuidados, llevando a discusiones que generan conflictos, pero que indefectiblemente necesitan llegar a un equilibrio.

4 LA INSPECCION—AUSCULTACION

El término auscultación (originalmente utilizado en Medicina) comenzó a ser utilizado en la Ingeniería con la intención de designar "acompañamiento" de una estructura que está en servicio, dentro de la cual se insiere la Inspección. Inspección es un término amplio que abarca la inspección visual y otras detalladas; pero de una forma común es entendida como inspección visual.

De forma general, la Auscultación de Represas cuenta como una herramienta la Inspección Visual. Y dentro de la Doctrina de Seguridad y Mantenimiento de Presas hay una variedad inmensa de Manuales, Rutinas y Publicaciones, en las cuales se procura orientar a los neófitos sobre la práctica de como cuidar de esas estructuras. Decurrente de eso surge una segunda pregunta:

☞ *¿Si hay un acervo grande de orientaciones, cual (es) es(son) la(s) razón(es) para que las Presas no tengan cuidado(s)-mantenimiento(s) adecuado(s)?*

5 LA CULTURA Y LA TRADICIÓN

Algunos Países, como el Japón[4]y[5] tratan de preservar el entorno de las represas con base en la cultura y tradición, revitalizando casas y propiedades, buscando establecer un equilibrio escénico.

6 EL TURISMO

6.1 *Generalidades*

El Turismo, abarca varias conceptuaciones, desde su etimología, su entendimiento como negocio (actual) y concepto/definición en la sociedad.

En el aspecto globalizado en que vivimos el Turismo implica[6] en negocios multimillonarios ($1*10^{12}$) de Dólares al año (datos de 2008), siendo que USA, Francia, España, China e Italia (no necesariamente en ese orden) son los Países que tienen el mayor flujo turístico

Los tipos de Turismo son diversos, desde el Turismo Agro-Cultural (vinos, bebidas y otros cultivos) al Culinario, Cultural, Eco-Turismo, Radical, Juegos, Náutico, Religiosos, Bélico y hasta, el Sexual.

Recientemente surgió el Turismo Tecnológico, que permite la visita de edificaciones/estructuras construidas con complexidades/dificultades propias y específicas o notorias.

Y, finalmente surgió el Turismo de las Catástrofes, como pro ejemplo el de Katrina o el de Tsunami.

6.2 *Ejemplos de Monumentos Turísticos*

Se pueden citar como Monumentos Turísticos los siguientes:

Tabla 1. Monumentos Turísticos.

Monumento	País	Flujo Turístico anual
Taj Mahal	India	Cerca de 3 millones
Coliseo	Italia	Mas de 1 millón
Alhambra	España	Mas de 3 millones
Torre Eiffel	Francia	Cerca de 7 millones
Estatua de la libertad	Estados unidos	Mas de 3 millones

6.3 *Ejemplos de Turismo Tecnológico*

Para ilustración, se pueden citar como Flujo de Turismo Tecnológico los siguiente puntos:

Tabla 2. Puntos Turísticos Tecnológicos.

Punto Turístico	País	Flujo Turístico anual
Centro Espacial-Kennedy- Cabo Cañaveral	Estados unidos	Mas de 1,5 millones
Hoover dam	Estados unidos	Mas de 1,0 millón
Itaipú dam	Brasil-paraguay	Cerca de 0,5 millón

Si se considera el costo de un "boleto" de US$ 10 para ingresar al Punto Turístico de menor Flujo (Itaipú Dam), tendríamos una recaudación de aproximadamente US$ 5.000.000,00 al año.

Eso tiene una relevancia desde el punto de vista Negocio-Turístico y, también genera servicios/empleos, pero ese no es el objetivo principal relacionado al Tema de Mantenimiento de Presas. Lo mas relevante es que al permitirse las Visitas Públicas, se establece un Potencial de Inspección, que induce a los Propietarios-Entidades Responsables por Guardar y Mantener las Obras a adoptar un nivel mas elevado de cuidados y de rutinas.

Se destaca que el Emprendimiento de Hoover, en la época de la Construcción (2ª mitad de la Década de 1930) costó cerca de 185 millones de US$ y que al fin de los años de 1980 la construcción de un edificio al lado de la Represa costó cerca de 180 millones de US$. Recientemente (durante esta década) la implantación de una ruta (The Hoover Dam Bypass Project) que incluye el Nuevo Puente del Colorado costará cerca de 234 millones de US$.

De todo esto viene la tercera y oportuna pregunta:

☞ *¿Si el Turismo y la Visitación Pública a un Bien Público es benéfica, cual es la razón para no adoptarla?*

7 LA INSERCION DE LOS PROYECTOS HIDRAULICOS EN EL CAMPO TURISTICO

No hay dudas de que la actividad Turística es una enorme fuente de empleos y de renda, que no cabe aquí debatir, no en tanto es saludable debatir la eventual inserción de los Proyectos Hidráulicos en el Campo Turístico, de tal forma que se pueda contar con:

- *Un potencial número de nuevos "Inspectores";*
- *Un nuevo potencial de profesionales para "Cuidar" del Bien Público;*
- *El conocimiento sobre las dificultades, desafios, cuidados y eventuales costos del Emprendimiento;*
- *La diseminación de ese conocimiento, adquirido en las visitas;*
- *El mejor entendimiento sobre el aprovechamiento hidráulico, e consecuentemente,*
- *La mejor manera de atingir un equilibrio correcto para el uso de los recursos de la naturaleza.*

8 LA INGENIERIA

Los Congresos, Publicaciones de Libros y Textos Técnicos, como así también la dinámica actual de la Internet, posibilitan a los técnicos una mejoría de las Técnicas de mantenimiento, observación, adecuando procedimientos, que casi siempre buscan sistemáticamente la minimización de Costos de Operación y Mantenimiento, pero que ni siempre inducen a una mejoría de la Calidad y de Seguridad.

De forma generalizada el conocimiento técnico burocratiza el sentido común y disminuye el entendimiento humano cuando se buscan soluciones globales, estableciendo un dogma común de que "*Los Ingenieros son malos Abogados de sus causas"!*

9 COMENTARIOS Y DISCUCIONES

Probablemente, el mayor desafío que envuelve el uso sustentable de los ríos y de sus áreas de influencia, es el de motivar a la población a conocer y evaluar sobre un Futuro Sustentable como también concordar sobre su conveniencia.

Las imposiciones y/o prohibiciones, como también impedir, históricamente solo dificultaron la comprensión y el entendimiento.

En estas situaciones se busca la cooperación de la inserción y de la divulgación que representa la necesidad!

Esto genera la cuarta pregunta:

☞ *¿Que se puede hacer ?*

Las Reglas de Organización y Planeamiento se tornan entonces, mas convenientes y necesarias. Y así tenemos:

Indagaciones	Concepto para debate
A quien cabe cuidar de esas represas-monumentos, o eses Bienes Públicos?	La entidad pública responsable y al Ciudadano con exigencias actualizadas e renovables
Se hay un gran acervo de orientaciones, cual(es) es(son) la(s) razón(es) para que as Represas no tengan un cuidado-mantenimiento adecuado?	Hacer cumplir!
Si el Turismo y la Visitación Pública a un Bien Público es benéfica, cual es la razón para su no adopción?	Adoptar!
Que se puede hacer?	Organizar y planificar la inserción!

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Influencia de la presencia de cenizas volantes en el conglomerante en la durabilidad de los hormigones de presas

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RESUMEN: Se pretende analizar la influencia que tiene en indicadores de durabilidad de los hormigones la presencia de las cenizas volantes en sus conglomerantes. Para ello se examina la influencia de las cenizas volantes en el principio y fin de fraguado de las pastas, en el calor de hidratación, en la presencia de $Ca(OH)_2$, en la demanda de agua para alcanzar un nivel de consistencia en morteros, en las resistencias a compresión y flexotracción de morteros, en la permeabilidad al oxigeno de morteros y en la estructura porosa de morteros.

1 INTRODUCCION

La evolución del diseño y de la construcción de las presas ha estado condicionada a lo largo de la historia por la del conocimiento científico y, de forma determinante, por la del nivel tecnológico. El proceso histórico del desarrollo de los hormigones masivos en el mundo se correlaciona directamente con el desarrollo tecnológico de la construcción de las presas en Estados Unidos.

- Antes de 1900: Periodo pretecnológico y de utilización de medios básicamente manuales.
- Años 1900–1930: Primeros pasos del desarrollo tecnológico.
- A partir de 1930: Periodo de rápido desarrollo de la tecnología específica de construcción de grandes presas.

El desarrollo tecnológico conlleva una mejora en los métodos y procesos constructivos y, consecuentemente, en el incremento de la velocidad de construcción, lo que ha obligado a su vez a actuar en tres frentes distintos:

- Utilización de conglomerantes de características adecuadas (bajo calor de hidratación)
- Utilización de técnicas de refrigeración del hormigón y/o de sus componentes
- Revisión de los criterios de diseño relacionados con las formas de puesta en obra.

Las presas de mampostería se ejecutaban habitualmente por tongadas horizontales de espesores comprendidos entre uno y dos metros según los casos y medios disponibles. La reducida cantidad de cemento utilizada, y por lo tanto del calor hidratación generado, y la lenta puesta en obra permitían la evacuación del calor generado sin ninguna repercusión.

Los problemas relacionados con los altos valores del calor de hidratación generado empezaron a surgir con la utilización del hormigón en masa. La técnica del hormigón desde sus inicios busco la solución del problema, y pronto se comprendió que ella se encontraba en la actuación sobre la fuente del problema o en la eliminación de sus consecuencias:

- Evitar una alta generación de calor de hidratación en la masa del hormigón mediante la utilización de cementos con bajo calor de hidratación, refrigerar los componentes del hormigón o la refrigerar la masa del hormigón mediante la circulación de agua por circuitos dejados al efecto en su interior.
- Establecer juntas de contracción que permitiesen la libre retracción térmica.

Las medidas contempladas en el primer método busca el control de la temperatura máxima del hormigón después del proceso de hidratación del conglomerante o una extracción rápida y controlada del calor de hidratación. En el segundo, mediante el señalamiento de grietas artificiales controladas, se pretende evitar las consecuencias descontroladas de la retracción térmica.

1.1 *Conglomerante fríos*

En el año 1932 se concluyó que el cemento Pórtland estaba formado por cuatro especies químicas fundamentales: los silicatos tricálcico y bicálcico, el aluminato tricálcico y el aluminoferrito tetracálcico, a los que acompañan cal libre, magnesia, sulfato cálcico y álcalis en menor cantidad, y que por lo tanto la diferencia entre los distintos cementos fabricados se debía a las distintas proporciones en los que intervienen estos elementos. Se demostró que el silicato tricálcico era el responsable del endurecimiento

Tabla 1. Calor de hidratación de los componentes del cemento Pórtland.

Elemento	Calor de hidratación (cal/gr)
Silicato tricálcico (SC_3)	120
Silicato bicálcico (SC_2)	62
Aluminato tricálcico (AC_3)	207
Aluminoferrito tetracálcico (AFC_4)	100
Cal libre	279
Magnesia	203

rápido; que una mayor proporción de silicato bicálcico influía en una menor resistencia inicial y en una resistencia final similar a la de los cementos rápidos; que el aluminato tricálcico era atacable por los sulfatos, y que el aluminoferrito tetracálcico era menos atacable que el resto de los componentes principales. Se determinaron los calores de hidratación de cada uno de los componentes.

Una de las formas de conseguir un bajo nivel del calor de hidratación en orden a limitar los esfuerzos térmicos es reducir en la medida de lo posible los contenidos en aluminato tricálcico y en silicato tricálcico.

En la búsqueda de conglomerantes de bajo calor de hidratación se empezaron a utilizar cementos en los que parte del Pórtland se sustituía por otros materiales con o sin propiedades puzolánicas (escorias o cenizas volantes).

1.1.1 *Sand-cement*

En 1893 una compañía de Dinamarca patentó una mezcla obtenida a base de moler conjuntamente arena silícea y cemento. Desde un primer momento se le asociaron algunas ventajas respecto al cemento Pórtland que se utilizaba habitualmente, lo que hizo se instalaran en Estados Unidos algunas fábricas que explotaron esta patente y que se construyeran en los últimos años del siglo XIX algunas presas con este conglomerante.

Debido al bajo precio de los cementos en Estados Unidos su uso decayó en los primeros años del siglo XX, hasta que el gran impulso que se le dio a las obras de riego a partir de 1905 hizo que se reconsiderase la cuestión. Estas obras de riego estaban situadas en zonas muy alejadas de las fábricas de cemento e incluso de vías de ferrocarril, lo que encarecía notablemente su precio final. El uso del sand-cement posibilitaba la reducción de volumen de cemento transportado. Se decidió su uso en las presas de Arrowock (1915) y de Elephant Butte (1916), consiguiéndose ahorros próximos al 30%. De la construcción de estas dos presas se dedujo:

- se obtenía un producto más impermeable que con el cemento Pórtland a igualdad de dosificación.
- la retracción hidráulica era más reducida.
- el fraguado era más lento y los incrementos de temperatura en la masa de hormigón menores.

A efectos resistentes se estableció una equivalencia entre el cemento Pórtland y el sand-cement: 275 kg de sand-cement con una presencia de clinker entre el 50 y el 55% equivalía a una dosificación de entre 230 y 250 kg de cemento.

La disminución del calor de hidratación fue fundamental para su utilización, pudiéndose considerar al sand-cement como precursor de los cementos fríos modernos.

En España se construyeron con sand-cement las importantes presas de Talarn (1916) de 86 metros de altura y la de Camarasa (1920) de 103 metros.

1.1.2 *Puzolanas*

Las puzolanas son materiales sílico o silicoaluminos que por sí solos poseen poco o nulo valor cementante, pero finamente dividido y en presencia de humedad y álcalis reaccionan formando compuestos con propiedades cementantes.

Las puzolanas tienen los mismos componentes que el cemento pero en distintas proporciones, lo que les confiere características diferenciales que hacen que su mezcla con ellos implique procesos de hidratación y endurecimiento distintos.

La Ceniza Volante (CV) es una puzolana que se obtiene por precipitación electrostática o por captación mecánica de los polvos que acompañan a los gases de combustión de los quemadores de centrales termoeléctricas alimentadas por carbones pulverizados.

Las primeras referencias significativas del uso de las cenizas volantes datan de 1946, siendo en el V Congreso ICOLD (1955) y en el VI Congreso (1958) cuando el Bureau of Reclamation presento resultados concluyentes sobre la adecuación del uso de las cenizas volantes como adición en los conglomerantes de los hormigones de presas. Tenía como inconveniente la menor resistencia a las heladas a edades tempranas, pero las ventajas eran de consideración:

- Resistencias algo menores a 28 días e iguales o mayores a partir de 90 días.
- Menor desprendimiento de calor, del orden del 40 al 50% inferior.
- Mayor impermeabilidad.
- Menor expansividad.
- Mayor facilidad de puesta en obra.

2 LOS HORMIGON DE PRESAS

En la construcción de presas deben emplearse materiales cuyas propiedades intrínsecas, su puesta

en obra y la evolución de sus características sean susceptibles de control.

El hormigón fresco es un estado durante el cual los granos de sus constituyentes -conglomerante, arena y grava- son susceptibles de tener movimientos independientes los unos respecto de los otros. En la práctica, el hormigón en este estado está expuesto a la segregación, pero ofrece la oportunidad de ser consolidado, minimizando su porosidad inicial. Para la comprensión y control del comportamiento del hormigón en este estado, se ha desarrollado la idea y los métodos de medición de la consistencia.

El hormigón endurecido es un sólido, un conglomerado, un medio poroso cohesivo y dinámico. La estructura de un sólido, en sentido general, se define como la relación entre la posición espacial de los elementos individuales primarios y las fuerzas de unión entre ellos. La interacción de unos elementos estructurales con otros o con el medio ambiente debe ser estudiada en los diferentes niveles estructurales del hormigón. Una clasificación de estos niveles puede ser:

1. El nivel atómico-molecular es el más elemental o primario de la composición estructural del hormigón. A este nivel corresponderían propiedades como:
   – Solubilidad en agua
   – Posibilidad de reaccionar con el agua o de resistir su acción
   – Estabilidad térmica
   – Atracción a diferentes clases de compuestos químicos
2. La superficie de la interfase entre las diversas partículas primarias determina el nivel siguiente en la complejidad estructural. La energía superficial de cada partícula condiciona su correspondiente conducta en relación con el resto de las partículas que la rodean.
3. En un tercer nivel están las formaciones de multipartículas y los huecos de aire. En este nivel, las propiedades del hormigón determinan su capacidad de resistir numerosas influencias agresivas, incluidos los ciclos de congelación y deshielo.
4. Nivel de elemento estructural condiciona la distribución de tensiones.

En el tercer nivel, formaciones de multipartículas y los huecos de aire, aparece la porosidad como característica intrínseca relevante del hormigón. Además, la resistencia, densidad y permeabilidad del hormigón se determinan sobre muestras cuyo nivel estructural es asimilable este.

2.1 *La porosidad del hormigón endurecido y su durabilidad*

Al fraguar el hormigón, como consecuencia del endurecimiento de la pasta de cemento, queda constituido como un material surcado por toda una serie de poros y capilares, transformando al hormigón en un material poroso.

El hormigón es un material de gran heterogeneidad y dinámico en función del medio ambiente. El carácter poroso del hormigón y la fase acuosa contenida en los poros, entre otros parámetros, juegan un papel fundamental en la evolución del hormigón y, por lo tanto, de sus características intrínsecas.

Los poros y la fase acuosa que aquéllos contienen constituyen un sistema termodinámicamente abierto en continua interacción, cuyos equilibrios fluctúan en función del medio ambiente exterior, humedad relativa, penetración de agentes etc. ... La composición química de la fase acuosa contenida en los poros regula los mecanismos de reacción.

La estructura de los poros y la configuración de las fisuras, así como el contenido de agua en las mismas, son factores que determinan la permeabilidad del hormigón que, a su vez, controla en gran medida su relación con el medio ambiente que lo rodea.

Este hecho es de gran importancia en orden a su durabilidad; por un lado, la acción de los agentes agresivos comienza en la superficie del hormigón, desarrollándose más tarde en el interior a través de los poros; por otro lado, la superficie interna de los poros es muchas veces mayor que la superficie exterior del hormigón, ampliando así drásticamente la superficie de actuación de los agentes agresivos.

En definitiva, la estructura de poros del hormigón determina en gran manera la intensidad de su interacción con un medio agresivo. Un hormigón con una estructura de poros inadecuada se ve, lógicamente, más afectado por la helada, por el ataque de aguas y suelos agresivos, por la penetración de cloruros, etc.

Un cuerpo poroso, como es el caso del hormigón, presenta dos tipos diferenciados de huecos: los que pueden ser accesibles o abiertos y con comunicación con el exterior, y los inaccesibles o cerrados y aislados del exterior.

– Porosidad total: Es la fracción del volumen total de hormigón ocupada por los poros.
– Porosidad abierta: Es la parte de la porosidad total que comprende los poros conectados con el exterior.
– Porosidad permeable: Es la parte de la porosidad abierta que corresponde a poros intercomunicados entre sí, permitiendo el paso de un fluido al interior del hormigón.
– Porosidad superficial en fondo de saco: Es la parte de porosidad abierta que corresponde a poros no comunicados con otros, no permitiendo el paso de fluido.
– Porosidad cerrada: Es la parte de la porosidad total que incluye los poros no conectados con el exterior y, por tanto, inaccesibles al medio agresivo externo.

Con vistas a la durabilidad del hormigón, interesan tres aspectos de su estructura de poros: el volumen total de poros, su distribución en los diferentes tamaños de poro y el grado de accesibilidad de los mismos desde el exterior y su interconexión con el interior.

2.1.1 *Datos hormigones cinco presas*

Se han llevado a cabo ensayos sobre muestras obtenidas en cinco presas. Se han determinado las permeabilidades al oxigeno y las porosidades totales abiertas de testigos sanos y deteriorados, entendiendo por deteriorados los testigos que tenían resistencia a compresión muy reducida.

Se observa como por encima de un nivel de permeabilidad al oxigeno y porosidad total abierta la mayoría de los hormigones están deteriorados.

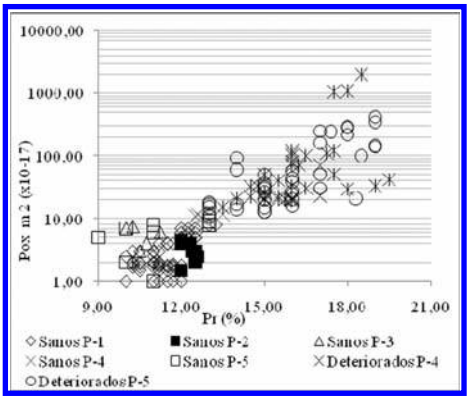

Figura 1. Relación permeabilidad oxigeno con porosidad total abierta de hormigones.

3 HIDRATACION DE CONGLOMERANTES MEZCLA DE CEMENTOS PORTLAND CON CENIZAS VOLANTES

La norma ASTM C 150 define al cemento Pórtland como un conglomerante hidráulico producido al pulverizar el clinker y que consiste esencialmente de silicatos de calcio, conjuntamente con una o más formas de sulfato de calcio. El clinker se produce por una mezcla de materiales sinterizados a aproximadamente 1400°C. Los principales óxidos presentes en la materia prima son CaO, SiO_2, Al_2O_3 y Fe_2O_3.

El fraguado y endurecimiento del cemento Pórtland ocurre como resultado de la reacción de hidratación entre los compuestos del clinker, el yeso y el agua, la cual genera calor que es conocido como calor de hidratación. Las reacciones se producen a velocidades diferentes y están influenciadas unas con otros.

El CSH es la principal fase de hidratación y la responsable de la resistencia mecánica de las pastas de cemento. La portlandita o hidróxido cálcico es el segundo producto más abundante en las pastas de cemento hidratado. La portlandita no aporta ningún beneficio a la capacidad mecánica de la pasta de cemento y puede incrementar los espacios porosos al lixiviarse y salir del hormigón, precipitándose en forma de carbonato cálcico. En un cemento Pórtland totalmente hidratado, la portlandita generada constituye de un 20 a un 25% del contenido sólido.

$CS + HO_2 \ldots CSH + Ca(OH)_2 + Calor$

La norma ASTM 618 define a las puzolanas como aquellos materiales sílico o silicoaluminos, los cuales por si solos poseen poco o nulo valor cementante, pero finamente dividido y en presencia de humedad reaccionan químicamente con la portlandita (hidróxido cálcico $Ca(OH)_2$) a temperatura ordinaria para formar compuestos con propiedades cementantes.

Cuando en presencia de agua reacciona una mezcla de cemento Pórtland con CV, se produce una reacción similar a la de acido-base, participando en ella por una parte la cal y los álcalis liberados en la hidratación del cemento Pórtland y por la otra los óxidos ($SiO_2 + Al_2O_3 + FeO_3$) de la puzolana, modificándose por ello los mecanismos de hidratación de los compuestos principales del cemento Pórtland, afectando al calor generado, contenido de portlandita, cantidad de agua combinada y grado de reacción de los componentes.

El principal producto de la reacción es la fase CSH, que es similar al producido en la reacción del agua con el cemento Pórtland y que contribuye a la generación de capacidades mecánicas, especialmente a edades avanzadas de hidratación. Parte de los álcalis que participan en la reacción, y que fueran liberados en la reacción de hidratación, quedan adsorbidos en la estructura de los CSH, lo que también contribuye a disminuir una posible reacción del árido en el hormigón.

$CV + Ca(OH)_2 + H_2O \ldots CSH + Calor$

En la reacción de hidratación del cemento Pórtland mezclado con cenizas volantes se producen productos de hidratación muy similares a los producidos con cemento Pórtland: CSH (silicato de calcio hidratado), C_2ASH_8 (hidrato de gelenita), $Ca(OH)_2$ (hidróxido cálcico), $CaCO_3$ (carbonato de calcio), etringita, hidratos de aluminato de calcio y monosulfoaluminato.

4 TRABAJO EXPERIMENTAL

El trabajo experimental que se presenta ha sido realizado dentro del Grupo de Trabajo sobre Presas de Hormigón Compactado con Rodillo (HCR) que en la Universidad de Cantabria desarrolla sus actividades desde 1982.

4.1 *Materiales utilizados*

4.1.1 *Cemento y cenizas volantes*

El cemento utilizado es un cemento pórtland tipo I/45 A. Según la norma UNE 80-301 es un conglomerante hidráulico constituido a base de clinker de pórland en proporción no menor del 95%, ni mayor del 99% en masa, y de componentes adicionales en proporción no mayor del 5% ni menor del 1% en masa. Estas proporciones no incluyen ni el regulador de fraguado ni los eventuales aditivos.

Las cenizas utilizadas son de las denominadas silicoaluminosas o de clase F con bajo contenido en cal, según la norma ASTM C618-85.

Los conglomerantes que son objeto de ensayo se obtienen por la sustitución de distintos porcentajes (%) en peso de cemento por CV.

4.1.2 *Arena*

Se utiliza arena normalizada CEN EN 196-1. La arena de referencia CEN es una arena natural de granos redondeados cuyo contenido de sílice es como mínimo el 98%.

4.2 *Ensayos realizados*

El fin del estudio es analizar la influencia de la presencia de cenizas volantes en el conglomerante en indicadores de durabilidad de los hormigones. Para ello se analiza su influencia en el principio y fin de fraguado, en el calor de hidratación generado, en la presencia de $Ca(OH)_2$ durante el proceso de hidratación, en la acción plastificante, en la capacidad mecánica y en su estructura porosa.

En primer lugar el análisis de la influencia en la capacidad mecánica y estructura porosa se realiza sobre morteros de consistencia normal, es decir con relación a/(c + CV) variable. Ello se hace por dos razones:

- Para tener en consideración toda la potencialidad de las cenizas volante, acción plastificante y puzolánica.
- Para que las pastas y morteros tengan la misma actitud de ser puestas en obra en el laboratorio y, por ello, tengan la misma porosidad inicial. La porosidad de una pasta o mortero es igual a la porosidad cementicia más la porosidad inicial. La porosidad inicial es la que tiene el material inmediatamente después de su puesta en obra, es decir, refleja el grado de adecuación del comportamiento de la mezcla en estado fresco y la puesta en obra. La porosidad cementicia es característica del tipo de conglomerante utilizado y de su dosificación. El mortero de consistencia normal (UNE 83-258) es el que después de ser sometido a la acción de la mesa de sacudidas, la torta tiene un diámetro medio de 151,3 mm (relación a/(c + CV) = 0,5.

Tabla 2. Composición en óxidos del cemento Portland y la ceniza volante.

Óxidos	Cemento Pórtland I/45 A (%)	CV (%)
SiO_2	18,27	48,55
Al_2O_3	5,83	25,45
Fe_2O_3	1,58	9,09
CaO	64,19	4,15
MgO	1,79	0,91
Na_2O	0,35	1,78
K_2O	0,80	2,36
SO_3	3,19	1,65
P.C.	3,38	4,65

Tabla 3. Granulometría de la arena normalizada CEN EN 196-1.

Tamiz (mm)	2,50	1,25	0,63	0,32	0,16
% pasa	100	80,7	44,1	28,4	10,4

4.1.3 *Nomenclatura*

Tabla 4 Nomenclatura.

c	Cemento
CV	Ceniza Volante
T	Tiempo
x% CV	Conglomerante con un x% de sustitución de cemento por ceniza volante en peso
Rcc	Resistencia compresión morteros consistencia constante
Rcv	Resistencia compresión morteros consistencia variable
Rfc	Resistencia flexotracción morteros consistencia constante
Rfv	Resistencia flexotracción morteros consistencia variable
Rc	Resistencia flexotracción morteros
Rf	Resistencia flexotracción morteros
Poxc	Permeabilidad al oxigeno morteros consistencia constante
Poxv	Permeabilidad al oxigeno morteros consistencia variable
Pox	Permeabilidad al oxigeno morteros
Prc	Porosidad abierta ensayos consistencia constante
Prv	Porosidad abierta ensayos consistencia variable

Es decir, una consistencia de 51,3%, ya que la consistencia es c (%) = Dm-100.

En segundo lugar se realiza a consistencia variable con relación a/(c + CV) = 0,5, con lo que se analiza exclusivamente la acción puzolánica y no la reductora de agua.

4.2.1 *Normativa y metodología de ensayo*

- Consistencia normal de pastas UNE 80-102.
- Principio y fin de fraguado de pastas UNE 80-102.
- Consistencia de morteros UNE 83-258.
- La cantidad de hidróxido cálcico en las pastas se determina mediante la técnica termogravimétrica, que determina las variaciones de masa que acontecen en una muestra en el curso de un calentamiento continuo. Se utiliza una balanza termogravimétrica marca Shimazdu TGA-50.
- Resistencia a compresión y flexotracción UNE 80-101.
- Porosimetría de mercurio en un Micromeritics AutoPoro II 9215 con dos compartimentos, uno para baja presión y una cámara de alta presión de paredes gruesas de acero inoxidable. La baja presión tiene por objeto extraer mediante una bomba de vacio la humedad, gases y vapores de la muestra, llenar de mercurio el penetrómetro (portamuestras), y hacer la intrusión inicial para determinar los macroporos con diámetros entre 30 y 6 µm. El ensayo a alta presión va desde la presión atmosférica (unos 15 psi) a 4200 atmosferas (unos 60000 psi) e induce la intrusión de mercurio, posibilitando la determinación de la distribución de poros con diámetros entre 6 y 0,003 µm.
- La permeabilidad al oxigeno se realiza aplicando el método UC. Este método está basado en:
  - Norma ASTM C-577-68 para la obtención de la permeabilidad de refractarios.
  - Recomendación PRE para la determinación de la permeabilidad a gases de productos refractarios.
  - Recomendación Cembureau para la medida de la permeabilidad al oxigeno del hormigón.
  - Recomendación del Departamento de Ingeniería Civil de la Universidad de Leeds para la determinación de la permeabilidad al oxigeno del hormigón.

4.3.2.4 Análisis de resultados

- Para una determinada porosidad total abierta a mayor volumen de poros <0,1 µm menor permeabilidad al oxigeno y a mayor volumen de poros >0,1 µm mayor permeabilidad al oxigeno.
- Los poros <0,1 µm parece tienen un componente importante de porosidad en fondo de

4.2.2 *Influencia de la presencia de cenizas volantes en el principio y fin de fraguado*

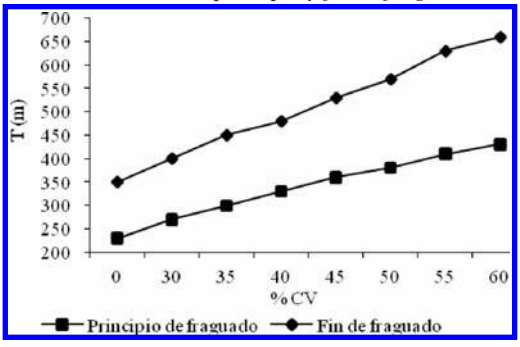

Figura 2. Influencia presencia CV en principio y fin de fraguado de pastas.

4.2.3 *Influencia de la presencia de cenizas volantes en el calor de hidratación*

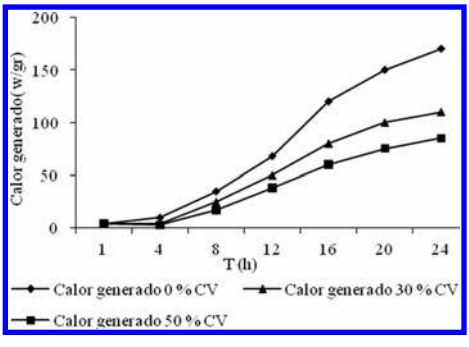

Figura 3. Influencia de la presencia CV en el calor de hidratación.

4.2.4 *Influencia de la presencia de cenizas volantes en la presencia de Ca (OH)$_2$ en el proceso de hidratación*

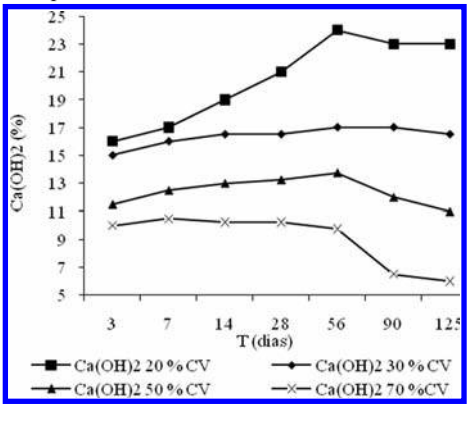

Figura 4. Influencia presencia CV en el contenido $Ca(OH)_2$.

4.2.5 *Influencia de la presencia de cenizas volantes en la demanda de agua en morteros de consistencia normal. Acción plastificante*

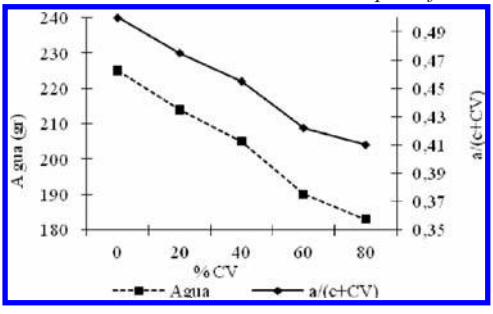

Figura 5. Influencia de la presencia de CV en la demanda agua en morteros normalizados. Acción plastificante.

4.2.6 *Influencia de la presencia de cenizas volantes en la resistencia a compresión de morteros*

4.2.6.1 Morteros consistencia constante

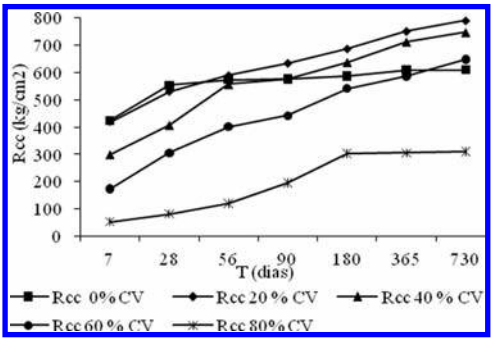

Figura 6. Evolución resistencia a compresión morteros consistencia constante.

4.2.6.2 Morteros consistencia variable

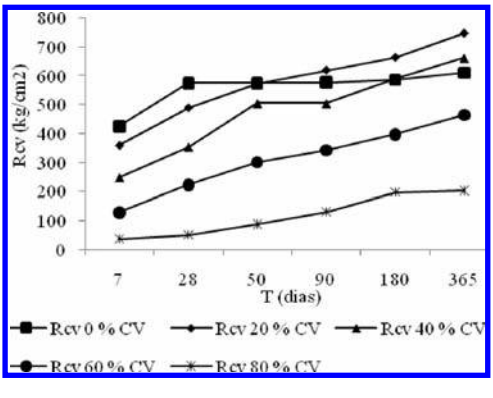

Figura 7. Evolución resistencia a compresión morteros consistencia variable.

4.2.7 *Influencia de la presencia de cenizas volantes en la resistencia a flexotracción*

4.2.7.1 Morteros consistencia constante

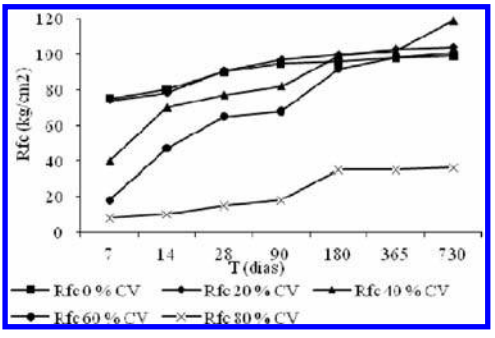

Figura 8. Evolución resistencia a flexotracción de morteros de consistencia constante.

4.2.7.2 Morteros consistencia variable

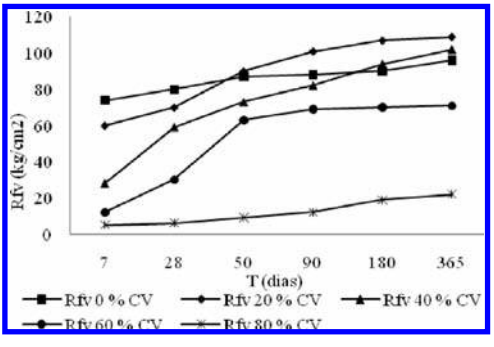

Figura 9. Evolución resistencia a flexotracción de morteros consistencia variable.

4.2.8 *Influencia de la presencia de cenizas volantes en la permeabilidad al oxigeno de morteros*

4.2.8.1 Morteros consistencia constante

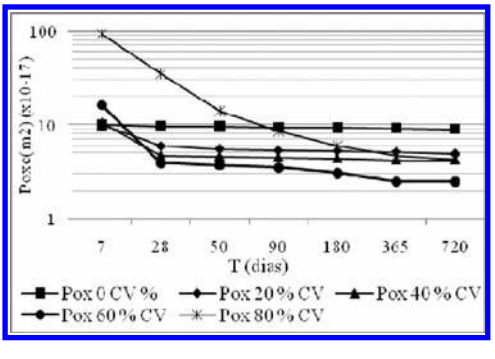

Figura 10. Evolución del la permeabilidad oxigeno de morteros consistencia constante.

4.2.8.2 Morteros consistencia variable

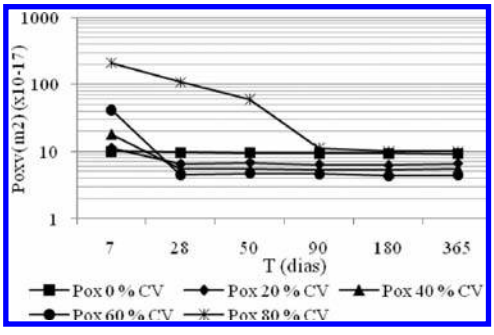

Figura 11. Evolución del la permeabilidad oxigeno de morteros consistencia variable.

4.2.9 *Influencia de la presencia de cenizas volantes en la estructura porosa de morteros*

4.2.9.1 Morteros consistencia constante

4.2.9.1.1 Porosidad total abierta

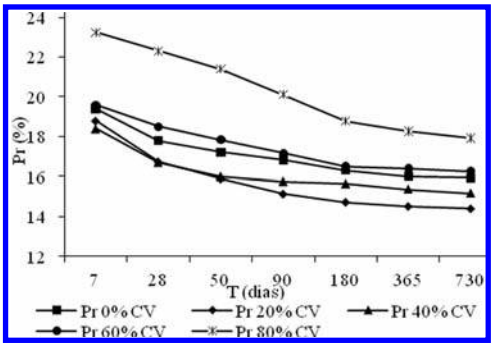

Figura 12. Evolución porosidad abierta mortero de consistencia constante.

4.2.9.1.2 Distribución tamaños de poros

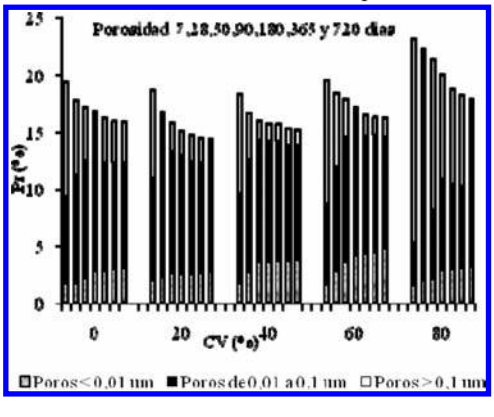

Figura 13. Evolución de la distribución del volumen de poros en morteros consistencia constante.

4.2.9.2 Morteros de consistencia variable

4.2.9.2.1 Porosidad total abierta

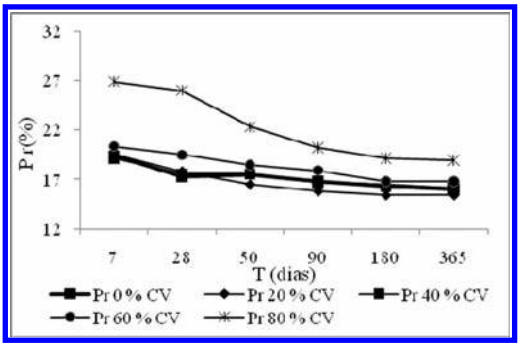

Figura 14. Evolución porosidad abierta mortero de consistencia variable.

4.2.9.2.2 Distribución tamaños de poros

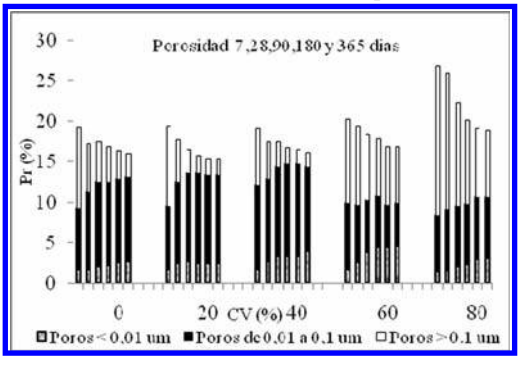

Figura 15. Evolución de la distribución del volumen de poros en morteros consistencia variable.

4.2.9.3 Análisis de datos estructura porosa

- Al aumentar la madurez, la presencia de cenizas volantes, hasta un alto porcentaje, produce una reducción de la porosidad total abierta.
- Al aumentar la madurez, la presencia de cenizas volantes, hasta un muy alto porcentaje, produce un incremento de los poros <0,01 µm.
- Al aumentar la madurez, la presencia de cenizas volantes en un conglomerante produce con generalidad un refinamiento de la red porosa, aumentando los poros <0,1 µm y reduciendo los poros >0,1 µm.

4.3 *Conclusiones*

4.3.1 *Generales*

Las cenizas volantes sílicoaluminosas (clase F) mezcladas con cementos Pórtland tipo I:

– Inducen un retraso en el principio y fin de fraguado.
– Provocan un efecto plastificante en la mezcla.
– Reducen notablemente el calor generado durante el proceso de hidratación.
– Tienen un claro efecto puzolánico, consumiendo portlandita y generando silicatos cálcicos hidratados.
– Potencian la estructura resistente hasta un muy alto porcentaje de presencia de CV.
– A altas madureces reducen la permeabilidad al oxigeno hasta muy alto porcentaje de presencia de CV.
– Reducen la porosidad total abierta hasta un alto porcentaje de presencia de CV.
– Refinan e interceptan los poros.

4.3.2 *Relación de la estructura porosa con la permeabilidad al oxigeno y las resistencias a compresión y flexotracción en morteros*

4.3.2.1 Relación porosidades abiertas con permeabilidad al oxigeno

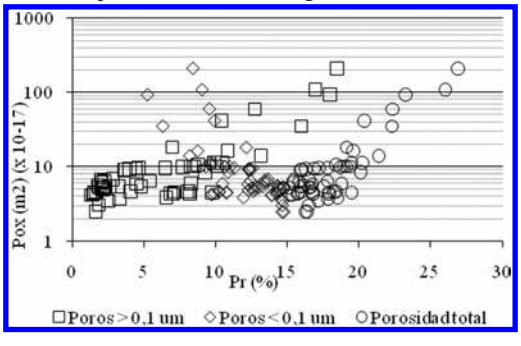

Figura 16. Relación de la permeabilidad al oxigeno con porosidades abiertas en morteros.

4.3.2.2 Relación de porosidades abiertas con la resistencia a compresión

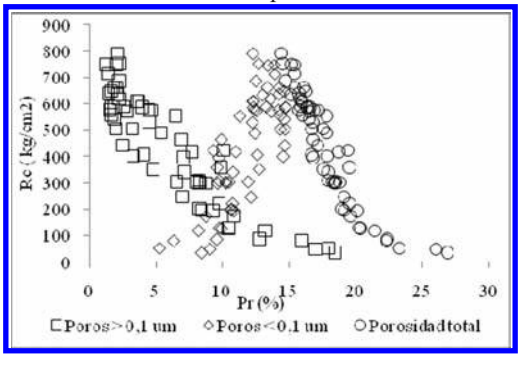

Figura 17. Relación de la resistencia a compresión con porosidades abiertas de morteros.

4.3.2.3 Relación de porosidades abiertas con la resistencia a flexotracción

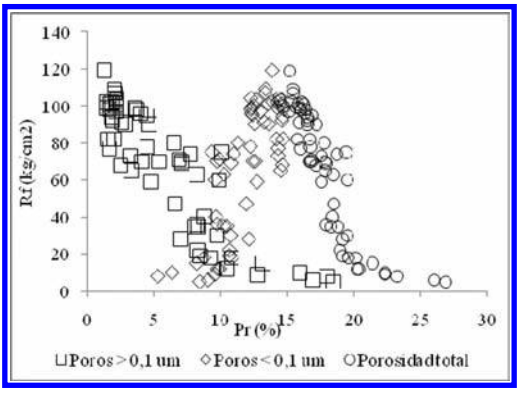

Figura 18. Relación de la resistencia a flexotracción con porosidades abiertas de morteros.

saco y los poros > 0,1 µm parece tienen una componente importante de porosidad permeable. Los productos procedentes de la reacción puzolánica disminuyen el tamaño de los poros y producen interrupciones en la estructura porosa.
– Para una determinada porosidad total abierta a mayor volumen de poros <0,1 µm, mayor resistencia a compresión y a mayor volumen de poros >0,1 µm, menor resistencia a compresión.
– Para una determinada porosidad total abierta a mayor volumen de poros <0,1 µm mayor resistencia a flexotracción y a mayor volumen de poros >0,1 µm menor resistencia a flexotracción.

5 LAS CENIZAS VOLANTES Y LA DURABILIDAD DE LOS HORMIGONES

Se entiende por durabilidad de un hormigón su capacidad para resistir a la acción ambiental, ataque químico, abrasión o cualquier otro proceso de deterioro. Los daños en las presas de hormigón se pueden relacionar con tres tipos de acciones: acciones químicas, acciones físicas y otras acciones. La acción química puede ser ejercida por agentes gaseosos, líquidos o sólidos, pero siempre en presencia de agua o humedad. Esta acción del agua está controlada por la permeabilidad, es decir, por el tamaño y continuidad de los poros y de las fisuras.

Se puede decir que las cenizas volantes mejoran la durabilidad de los hormigones de presas por:

– Tienen un efecto plastificante y reducen la demanda de agua para alcanzar un determinado nivel de manejabilidad.

– Reducen el calor de hidratación y por ello la posibilidad de fisuración.
– El efecto puzolánico hace reducir la presencia de $Ca(OH)_2$, que es el componente más inestable de los elementos generados en el proceso de hidratación. Así mismo, reduce la posibilidad de una eventual reacción álcali—árido al disminuir su presencia en el medio.
– A altas madureces la presencia de CV en el conglomerante potencia la estructura resistente hasta un alto porcentaje de presencia, y con ello se reduce la posibilidad de fisuración.
– A altas madureces la presencia de CV en el conglomerante reduce la permeabilidad hasta un muy alto porcentaje, y con ello la posibilidad de acceso a su interior de elementos nocivos.

Groundwork and periodic maintenance

Applicable legislation and standards

Groundwork and periodic maintenance

Applicable legislation and standards

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Consideraciones sobre el control y gestión de las presas y balsas de titularidad no estatal dentro de los organismos de cuenca intercomunitarios

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ABSTRACT: The hydraulic norms give to the basin authorities many responsibilities about inspection of water supplies, and also about the public works in relation with them, specially dams and pools. The Spanish safety laws are numerous, but they don't constitute a homogeneous framework. This generates difficulties to applicate them to similar situations. From the point of view of the basin authorities in the Ministerio de Medio Ambiente, y Medio Rural y Marino, there is a difference between national public works, and supplies of another public organisms or private owners. The problems have grown up since the publication of the *Real Decreto 9/2008*, which modifies the previous *Reglamento del Dominio Público Hidráulico*. It demands that the hydraulic authorities do an exam about the dams´ situation and current laws in Spain, and prepare answers to different safety management questions.

1 INTRODUCCIÓN

El vigente *Texto Refundido de la Ley de Aguas* (*Real Decreto Legislativo 1/2001, de 20 de julio*) señala en su art. 24, como parte de las competencias atribuidas a los organismos de cuenca:

- *El otorgamiento de autorizaciones y concesiones referentes al dominio público hidráulico, salvo las relativas a las obras y actuaciones de interés general del Estado, que corresponderán al Ministerio de Medio Ambiente.*
- *La inspección y vigilancia del cumplimiento de las condiciones de concesiones y autorizaciones relativas al dominio público hidráulico.*

Por otro lado, el *Real Decreto 984/1989, de 28 de julio*, en su art. 4, atribuye a las comisarías de aguas de los anteriores, entre otras, las siguientes facultades:

- *Las propuestas de otorgamiento de concesiones y autorizaciones referentes a las aguas y cauces de dominio público hidráulico, así como las de establecimiento de servidumbre, deslindes y modulaciones.*
- *La inspección y vigilancia de las obras derivadas de concesiones y autorizaciones de dominio público hidráulico.*
- *La inspección y vigilancia de las explotaciones de todos los aprovechamientos de aguas públicas, cualquiera que sea su titularidad y el régimen jurídico al que estén acogidos.*

Dentro de tales aprovechamientos, y especialmente en los que afectan a las aguas superficiales, puede darse la necesidad de utilizar obras de regulación y/o derivación de caudales, entre las cuales destacan las presas y las balsas, incluyendo en las primeras los llamados *azudes*, considerados como aquellas obras situadas en cauces de cualquier tipo, de escasa altura y capacidad de embalse, cuya función suele ser la de derivar aguas hacia un canal u otra infraestructura, más que almacenarlas en su propio vaso.

Si bien a todo aprovechamiento de aguas que disponga de alguna infraestructura de esta clase le son aplicables las disposiciones legales generales relativas a los usos de las aguas, debe recordarse que también existen ciertas normas que afectan particularmente a tales obras, con el fin de garantizar su seguridad en lo referente a posibles daños a terceros. En cualquier caso debe considerarse que el cumplimiento de la normativa específica de presas y balsas no justifica por sí misma la existencia de ninguna de ellas, si no es como parte de una explotación de aguas que se encuentre debidamente legalizada, ni exime del cumplimiento de la legislación aplicable a la anterior.

Como resulta conocido, la normativa de presas y embalses ha seguido en España un desarrollo irregular durante los últimos tiempos. La disposición fundamental y única durante muchos años -vigente aún en la actualidad-ha sido la *Instrucción para el Proyecto, Construcción y Explotación de Grandes Presas* (IGP), la cual fue

seguida por el *Reglamento Técnico sobre Seguridad de Presas y Embalses* (RTSPE), que no sustituyó totalmente a la primera, sino que vino a introducir una tratamiento diferenciador entre obras basado, no en aspectos de índole técnica, sino en criterios de tipo administrativo, con los consiguientes agravios comparativos que podían derivarse para obras con características constructivas e incidencia sobre el entorno similares.

Esta situación, complicada por el hecho de que existen infraestructuras que quedan fuera del ámbito de aplicación de ambas disposiciones, así como por el rango legal de las mismas, que excluía la existencia de unos medios coercitivos claros para las situaciones en las que se incumpliesen sus contenidos, motivó que durante bastante tiempo se trabajase sobre la posibilidad de publicar una ley que unificase de forma definitiva los criterios aplicables, y que tuviese el nivel jurídico necesario para facilitar su puesta en práctica. Sin embargo, ha sido mediante la aparición del *Real Decreto 9/2008, de 11 de enero*, cuando se ha producido la modificación del escenario existente, si bien introduciendo las normas de seguridad de presas, embalses y balsas como un nuevo título dentro del vigente *Reglamento del Dominio Público Hidráulico (Real Decreto 849/1986, de 11 de abril)*.

Como marco legislativo de referencia para la presente comunicación se considerarán las siguientes disposiciones:

- *Real Decreto 9/2008, de 11 de enero, por el que se modifica el Reglamento del Dominio Público Hidráulico, aprobado por el Real Decreto 849/1986, de 11 de abril.*
- *Real Decreto 984/1989, de 28 de julio, por el que se determina la estructura orgánica dependiente de la presidencia de las confederaciones hidrográficas.*
- *Directriz Básica de Planificación de Protección Civil ante el Riesgo de Inundaciones (Acuerdo del Consejo de Ministros de 9 de diciembre de 1994).*
- *Reglamento Técnico sobre Seguridad de Presas y Embalses (Orden Ministerial de 12 de marzo de 1996).*
- *Instrucción para el Proyecto, Construcción y Explotación de Grandes Presas (Orden Ministerial de 31 de marzo de 1967).*
- *Instrucción Técnica Complementaria 08.02.01 del capítulo XII del Reglamento General de Normas Básicas de Seguridad Minera "Depósitos de lodos en procesos de tratamiento de industrias extractivas" (Orden de 26 de abril de 2000).*

En el presente documento, y mientras no se indique otra cosa, debe entenderse que las alusiones a presas y embalses se refieren a obras que captan, derivan o regulan aguas, y que estarían sometidas como aprovechamientos, por tanto, al control de las autoridades en materia hidráulica. No se alude -en principio- a las infraestructuras que almacenan residuos, fangos o vertidos resultantes de explotaciones mineras, las cuales disponen de normativa específica, y dependen–en principio- de otras administraciones públicas diferentes de la hidráulica. No obstante, y como se verá en su momento, hay cuestiones en las que sí deben considerarse éstas últimas.

2 ASPECTOS A CONSIDERAR

Con respecto a la asunción y puesta en práctica de las obligaciones atribuidas a los organismos de cuenca intercomunitarios –y dentro de ellos, a las comisarías de aguas- en materia de presas y embalses no estatales, entendemos que pueden plantarse varias cuestiones generales que precisarán de la toma de decisiones, no sólo por parte de los dirigentes de los organismos cuenca, sino posiblemente también en los servicios centrales del Ministerio de Medio Ambiente, y Medio Rural y Marino, del cual dependen, e incluso en otras administraciones. Tales cuestiones se enuncian a continuación, de modo no preferente, y sin perjuicio de un desarrollo más preciso en apartados posteriores:

- Asignación de competencias relativas a la seguridad de presas y embalses a los diferentes organismos públicos.
- Asignación de las tareas correspondientes a las comisarías de aguas entre sus distintas unidades.
- Comprobar la situación de las diferentes obras existentes desde el punto de vista de los requisitos de seguridad.
- Definir los trabajos futuros a ejecutar por las comisarías de aguas en materia de seguridad.
- Necesidades de apoyos externos para atender las obligaciones de las comisarías de aguas.

3 ASIGNACIÓN DE COMPETENCIAS RELATIVAS A LA SEGURIDAD DE PRESAS Y EMBALSES

En este punto es necesario que se especifiquen con claridad las atribuciones que le corresponden a cada organismo, y debe hacerse de forma ineludible para cualquier planteamiento coherente de las actividades que corresponda realizar en el futuro. De modo resumido, se expone a continuación una tabla con la posible distribución de las mismas, basada principalmente en lo recogido en el art. 362 del *Real Decreto 9/2008* y en el art. 4 del *Real Decreto 984/1989*, si bien conviene señalar

que la relación que se presenta no coincide con las previsiones que se hacen sobre este particular en el *Reglamento Técnico de 1996* o en la *Instrucción de Grandes Presas de 1967*, las cuales mantienen aún su vigencia, en los términos previstos en la disposición transitoria primera del *Real Decreto 9/2008*:

	Organismos			
Competencias	OO.CC.	C.N.P.C.	MARM	CC.AA.
Clasificación en función del riesgo potencial			X	X
Informes sobre propuestas de clasificación	X		X	X
Informes sobre proyectos, cambios de fase o concesiones.	X		X	X
Inspecciones de construcción e informes de seguridad.	X		X	X
Informe de normas de explotación.			X	X
Aprobación de normas de explotación.			X	X
Informe de planes de emergencia.		X	X	X
Aprobación de planes de emergencia.			X	X
Evaluación de revisiones e informes de seguridad.	X		X	X
Condiciones de explotación y órdenes de vaciado.	X		X	X
Vigilancia de obligaciones sobre seguridad.	X		X	X
Registro de Seguridad de Presas y Embalses.	X		X	X

OO.CC.: Organismos de cuenca.
C.N.P.C.: Comisión Nacional de Protección Civil.
MARM: Ministerio de Medio Ambiente, y Medio Rural y Marino.
CC.AA.: Comunidades autónomas.

Como puede comprobarse, la situación resulta compleja por la posible concurrencia de diversos organismos para una misma actividad, sin que pueda reducirse exclusivamente a las que se refieren a la administración estatal frente a la autonómica, sino que, incluso dentro de la primera, también pueden darse tales indeterminaciones entre los servicios centrales del Ministerio de Medio Ambiente, y Medio Rural y Marino, y los organismos de cuenca adscritos al mismo. Por todo ello es necesario que existan unas normas de actuación y protocolos de relación muy claros entre los diferentes servicios implicados.

Merece la pena llamar la atención de manera especial –entre otras- sobre la paradoja que se crea en lo referente a las inspecciones de construcción e informes sobre seguridad, así como en la imposición de limitaciones en la explotación y órdenes de vaciado. Todas estas competencias, según se indicaba en la introducción de este documento, han estado tradicionalmente en manos de los organismos de cuenca debido a su papel de gestores del dominio público hidráulico, y responsables de la vigilancia y control de los aprovechamientos relativos al anterior, así como de las obras pertenecientes a los mismos, todo ello sin perjuicio de la intervención en determinadas cuestiones –como los informes sobre los proyectos de presas- de los servicios centrales ministeriales a los que los organismos de cuenca intercomunitarios se encuentran adscritos.

Esta situación, sin embargo, queda alterada tras la entrada en vigor del *Real Decreto 9/2008*, que en la redacción dada al art. 360.2 del *Reglamento del Dominio Público Hidráulico* impone que … *las comunidades autónomas designarán a los órganos competentes en materia de seguridad en relación con las presas, embalses y balsas situados en el dominio público hidráulico cuya gestión les corresponda, y en todo caso con las presas, embalses y balsas ubicados fuera del dominio público hidráulico*. En consecuencia, puede darse el caso de aprovechamientos de aguas públicas cuyo otorgamiento y policía conjunta corresponde a la administración estatal, pero con obras situadas fuera del dominio público hidráulico cuyo control y vigilancia se encomiendan a las autoridades autonómicas.

Otro tanto puede decirse respecto a la clasificación de presas y balsas en función del riesgo potencial, que está atribuida de manera exclusiva a la Dirección General de Obras Hidráulicas del Ministerio de Medio Ambiente, y Medio Rural y Marino (actualmente, Dirección General del Agua) por la *Directriz Básica de Planificación de Protección Civil ante el riesgo de inundaciones* en su punto 3.5.1.3, sin que dicha norma haya sido derogada por el *Real Decreto 9/2008*. Y por otra parte, también hay contradicciones sobre la obligatoriedad

de realizar la propuesta de clasificación, ya que en la *Directriz Básica*–no derogada ni modificada, al menos de forma expresa- se imponía la misma a todo tipo de obras sin distinción, mientras que el *Real Decreto 9/2008* sólo se exige para presas y balsas de altura superior a 5 metros, o de capacidad de embalse mayor de 100.000 m^3, cuando ya se ha venido aplicando el criterio anterior durante bastantes años.

De forma resumida, según el cuadro expuesto, y a falta de que haya un pronunciamiento definitivo por parte de la autoridad competente, se considera que las obligaciones que afectarían a los organismos de cuenca con arreglo al *Real Decreto 9/2008* -no necesariamente de forma exclusiva-son las siguientes:

- *Informar los proyectos y propuestas de clasificación, así como las circunstancias concretas que se presenten en el momento de proceder a un cambio de fase o etapa en la vida de la presa, o de producirse el otorgamiento o la renovación de la concesión.*
- *Inspeccionar la construcción de nuevas presas, informando sobre el cumplimiento de los requisitos de seguridad exigidos en el proyecto.*
- *Evaluar el contenido de las revisiones de seguridad y de los informes de seguridad.*
- *Establecer, por razones de seguridad, condicionantes a la explotación ordinaria y ordenar vaciados totales o parciales.*
- *Velar por el cumplimiento de todas y cada una de las obligaciones que en materia de seguridad corresponden al titular de la presa.*
- *Mantener actualizado el Registro de Seguridad de Presas y Embalses.*

4 ASIGNACIÓN DE LAS TAREAS CORRESPONDIENTES A LAS COMISARÍAS DE AGUAS ENTRE SUS DISTINTAS UNIDADES

Una vez que queden establecidas con claridad las competencias que deben asumir las comisarías de aguas en su condición de unidades encargadas de la policía de los aprovechamientos hidráulicos, será necesario distribuir los trabajos que requiera el ejercicio de las mismas entre las diferentes áreas y servicios de que consta aquélla. Sobre este particular conviene plantear las siguientes reflexiones:

- Según se ha comentado al principio de esta comunicación, las presas, embalses y balsas son obras cuya existencia debe justificarse como parte de un aprovechamiento de aguas, y por ello, el cumplimiento de los requisitos legales para garantizar su seguridad ha de verse como una parte dentro del condicionado general que se imponga en el título de derecho correspondiente. Sentado este punto de partida, parece razonable entender que la unidad encargada de tramitar las concesiones o autorizaciones de uso de las aguas, y de vigilar el cumplimiento de las imposiciones recogidas en las mismos, sea también la competente para atender a los asuntos relativos a la seguridad de las obras a las que afecten. Al anterior razonamiento podría responderse que tal asunción dentro de la unidad encargada de la gestión del dominio público hidráulico implicaría una carga de trabajo y responsabilidad importante sobre la misma, y además con una componente técnica elevada -no exclusivamente administrativa-, que podrían justificar el que tales responsabilidades se encomendasen a otra unidad diferente, ya fuera de las existentes en el organismo de cuenca, o incluso creada *ex profeso*. En todo caso, y si así se estableciese, entendemos fundamentales tres requisitos:
  - La unidad en cuestión precisará una importante capacidad para realizar estudios y trabajos de tipo técnico, tanto en cuanto a medios materiales, como humanos, incluyendo apoyos externos cuando se requieran.
  - Debe lograrse un buen nivel de coordinación con las unidades encargadas de la gestión del dominio público hidráulico.
  - Aquellas decisiones y resoluciones referentes a la seguridad de las obras que puedan incidir sobre cuestiones relativas al cumplimiento de las condiciones generales de las concesiones o autorizaciones, así como a la gestión del dominio público hidráulico han de ser tomadas contando con el conocimiento y opinión de las unidades encargadas de tramitar los títulos de uso de las aguas, y viceversa.
- La unidad encargada de los aspectos relativos a la seguridad de las presas debería entender también de los asuntos relativos a las inundaciones y daños que se produzcan como consecuencia de los accidentes o averías en las mismas. Según esto, habría que determinar si igualmente ha de hacerse cargo de las actuaciones relativas a avenidas que tengan origen distinto, en caso de estar segregada de las unidades de gestión del dominio público hidráulico, lo cual no parece, en principio, un planteamiento lógico.
- La obligación de crear y mantener el Registro de Seguridad de Presas y Embalses dentro de cada organismo de cuenca abre ciertos interrogantes, ya que dentro de cada demarcación hidrográfica -al menos, en las intercomunitarias- coexisten presas de titularidad estatal, controladas por la dirección técnica correspondiente del organismo, junto con las de otros propietarios, que quedan bajo responsabilidad de la comisaría de aguas. Por ello, será necesario establecer a cuál de las

dos anteriores unidades debe encomendarse dicha tarea. En atención al art. 4, apartados d) y e), del *Real Decreto 984/1989*, parece que serían las comisarías de aguas las indicadas para hacerse responsables del Registro de Seguridad; de ser así, ha de decidirse si ello requiere la creación de un servicio específico, con un esquema similar al seguido desde hace años para la llevanza del Registro de Aguas Públicas y el Catálogo de Aguas Privadas, o bien podría encomendarse al mismo servicio que se encargue de los anteriores, considerando el nexo que debe existir entre la información de cada obra, y la del aprovechamiento del que forme parte. En este sentido, y especialmente si se opta por establecer servicios separados, será necesario asegurar la coherencia entre los datos de ambos registros.

5 TRABAJOS FUTUROS A EJECUTAR POR LAS COMISARÍAS DE AGUAS EN MATERIA DE SEGURIDAD

Conviene exponer ahora, siquiera de forma somera, cuáles pueden ser las tareas a abordar en el futuro por las comisarías de aguas, dejando aparte la asignación que de las mismas se haga entre las unidades que la componen, sobre lo que ya se ha hablado en el punto anterior, bien entendido que podrían darse cambios en los próximos años, una vez que entren en vigor las normas técnicas de seguridad a las que se refiere el art. 364 del *Real Decreto 9/2008*. Es necesario considerar que estas actuaciones no agotan el papel que le corresponde a las comisarías de aguas respecto de las presas y balsas existentes dentro de la cuenca, y que han de ir paralelas a las generales de inspección y vigilancia de obras y aprovechamientos a las que se ha hecho alusión en puntos anteriores de este documento:

a. *Determinación del estado en que se encuentran las obras existentes desde el punto de vista legal*.- Se debería elaborar una relación de las presas y balsas afectas a expedientes de usos del dominio público hidráulico, con el fin de conocer cuáles se hallan en situación irregular, y cuáles no. Para ello no bastará sólo con asociar cada obra a su expediente, sino que se deberá comprobar si el mismo contempla la existencia de aquélla, y si sus características se ajustan a lo previsto en el título. Esto podría traer como consecuencia la necesidad de que la comisaría de aguas incoe procedimientos de modificación de características concesionales, sancionadores, e incluso de extinción de derechos.
b. *Tramitación las propuestas de clasificación pendientes*.- Se deberá determinar el protocolo necesario para que una propuesta sea admitida para su trámite, aclarando las siguientes cuestiones:
   - Organismos competentes de las administraciones autonómicas, en aplicación del art. 360.2 del *Real Decreto 9/2008*.
   - Servicio al que debe remitir el interesado la documentación, ya que se ha dado algún caso en que la misma ha llegado a poder del organismo competente para clasificar sin que se tuviera constancia en la comisaría de aguas de la cuenca.
   - Contenido mínimo de la propuesta a formular, especialmente en cuanto a la obligatoriedad o no de incluir documentación técnica que justifique la categoría solicitada, así como del cargo y titulación de la persona que deba suscribir la propuesta y, en su caso, la documentación que se acompañe.
   - Forma de dar traslado al solicitante de la resolución o trámites a cumplimentar, bien directamente por el organismo que resuelva, o a través de la comisaría de aguas.

     Además de lo anterior, se ha de establecer la forma de actuar con aquellas obras que cuenten con título legal, o formen parte de un expediente de legalización en trámite, y no hayan formulado la correspondiente propuesta. En primera instancia, bastaría con efectuar el requerimiento oportuno a los titulares por parte de las comisarías de aguas, pero considerando que la clasificación no ha de plantearse con obras en situación irregular, ya que ello podría dar carta de naturaleza a actuaciones que no sean justificables desde el punto de vista jurídico.
c. *Comprobar la existencia y situación de las normas de explotación y planes de emergencia*.- Al menos en aquellas obras que sean consideradas como *grandes presas*, o que hayan resultado clasificadas en categoría A o B estarán obligadas a presentar a la administración los documentos anteriormente citados, por lo que habrá que establecer determinaciones y protocolos similares a los que se han expuesto para las propuestas de clasificación, a fin de dar cumplimiento a tales obligaciones.

   Estas necesidades vienen impuestas en aplicación de lo previsto en la *Directriz Básica de Planificación de Protección Civil* y en el *Reglamento Técnico de 1996*. En la primera de ellas sólo se alude a los planes de emergencia, exigibles a las presas de categoría A o B, mientras que en el segundo se incluyen también las normas de explotación, de las cuales debería disponer toda obra a la que se aplique el mismo, lo que afectaría también a las *grandes presas*, según la definición de su art. 3.1, aunque no se clasifiquen como A o B. Conviene señalar que las definiciones de *gran presa* que aparecen en el *Reglamento Técnico de 1996*, en la *Instrucción*

de Grandes Presas de 1967, y en el *Real Decreto 9/2008* no son plenamente concordantes entre sí, lo que puede arrojar dudas sobre la aplicación a determinados casos.

Además de esto, y cuando se produzca la aprobación de los planes de emergencia correspondientes, será necesario participar en los comités de implantación de los mismos, conforme a los señalado en el *Acuerdo de la permanente de la Comisión Nacional de Protección Civil sobre criterios para facilitar la implantación material efectiva de los planes de emergencia de presas, de 30 de enero de 2003*, que establece la obligatoriedad de que en cada uno de tales comités figure un representante del organismo de cuenca afectado, además de otros pertenecientes a los servicios de protección civil, a las comunidades autónomas, y a la propiedad de la obra.

d. *Puesta al día de los archivos técnicos de las obras*.- De acuerdo a lo previsto en la *Instrucción de Grandes Presas de 1967* y en el *Reglamento Técnico de 1996*, deberá existir un archivo técnico en cada una de las obras a las que se apliquen las citadas disposiciones. Si bien es obligación del titular su elaboración y mantenimiento, será conveniente que la administración supervise el contenido del mismo, así como que se prevea la subsanación cuando dichos archivos no hayan sido formados, situación que previsiblemente se dará entre muchos de los titulares no estatales.
e. *Comprobación de las revisiones y análisis de seguridad de las obras*.- Deberían hacerse por los titulares, de conformidad y en los plazos previstos en el *Reglamento Técnico de 1996*, si bien el art. 33.4 contempla la posibilidad de que sean realizadas directamente por la propia administración. Hay que señalar la existencia de una contradicción a este respecto entre los puntos 1 y 2 de la disposición transitoria primera del *Real Decreto 9/2008*, ya que el primero remite a la aplicación del citado reglamento en tanto se aprueben las normas técnicas de seguridad previstas en el art. 364, mientras el segundo señala que la primera revisión –cuando aún no se haya realizado-se efectuará una vez publicada y de acuerdo a la norma técnica correspondiente.
f. *Inventario y localización de obras existentes*.- Independientemente de la información que corresponda incluir en el Inventario de Presas Españolas, cuyo mantenimiento es competencia de los servicios centrales del ministerio, cada organismo de cuenca precisará realizar un seguimiento constante de las obras ubicadas dentro de su ámbito territorial. Los datos procederán de los expedientes tramitados -tanto de legalización como sancionadores, en su caso- y de los informes redactados por las unidades encargadas de la vigilancia del dominio público hidráulico, pero conviene que sean complementados mediante barridos periódicos y sistemáticos del terreno aplicando las tecnologías de teledetección disponibles, así como con inspecciones *in situ* por equipos especializados.
g. *Control de las obras excluidas del ámbito de la normativa sobre seguridad*.- Con excepción de la *Directriz Básica de Planificación de Protección Civil*, el resto de las disposiciones legales específicas sobre presas y balsas de agua no cubren en su ámbito de aplicación, -ni individual, ni conjuntamente- a todas las obras existentes, sino que dejan fuera a aquéllas que, en atención a su tamaño y volumen de embalse, o a su menor riesgo potencial, se consideran como de menor importancia y peligrosidad. No obstante, el elevado número de las mismas, la gestión de los recursos hídricos almacenados, así como la posibilidad de que en su entorno existan o aparezcan en un momento dado elementos o infraestructuras que puedan quedar en situación de riesgo, hacen aconsejable estudiar la posibilidad de que por parte del organismo de cuenca se establezcan medidas de control y vigilancia complementarias de las establecidas por la legislación general ya descrita, o bien se eleve alguna propuesta en este sentido a las instituciones competentes en la administración central.

Otro aspecto que también es digno de ser recogido en este apartado es el que se refiere a las presas, balsas y embalses dedicados al almacenamiento de residuos y estériles mineros, a las cuales, y según la nueva disposición adicional que aparece como punto ocho del *Real Decreto 9/2008*, les será aplicable la legislación específica del sector minero; no obstante lo anterior, conviene recordar que no son raras las infraestructuras mineras que se construyen sobre terrenos de dominio público hidráulico, que forman parte de aprovechamientos en los que coexisten con obras de regulación de aguas limpias, o que podrían afectar al dominio público hidráulico con sus vertidos y lixiviados, lo cual hace que la administración hidráulica no pueda considerarse ajena a su situación y funcionamiento, siquiera sea por sus obligaciones genéricas de administración y control del señalado dominio público. Por ello, será necesario determinar con exactitud cuáles son las competencias que corresponden, tanto a la administración minera como a la hidráulica, con relación a este tipo de obras.

De la normativa específica del sector minero conviene recordar la *Instrucción Técnica Complementaria 08.02.01 del capítulo XII del Reglamento General de Normas Básicas de Seguridad Minera "Depósitos de lodos en procesos*

de tratamiento de industrias extractivas" (Orden de 26 de abril de 2000, del Ministerio de Industria y Energía), que podría considerarse como el equivalente -dentro del sector de la minería- al *Reglamento Técnico de 1996*. También hay que tener presente que las competencias sobre explotaciones mineras se encuentran transferidas en la actualidad a las administraciones autonómicas, lo que implica que éstas pueden imponer también disposiciones cuyo ámbito de aplicación estaría restringido a su propio territorio; un ejemplo de ello es el *Decreto 281/2002, de 12 de noviembre, por el que se regula el régimen de autorización y control de los depósitos de efluentes líquidos o de lodos procedentes de actividades industriales, mineras y agrarias*, publicado por la Junta de Andalucía (Consejería de Presidencia).

h. *Organización de las tareas de explotación por los titulares, y nombramiento de directores*.- Tanto en la *Instrucción de Grandes Presas de 1967*, como en el *Reglamento Técnico de 1996*, se contempla la obligación de que las obras a las que afectan sean construidas y explotadas bajo el control de un equipo de profesionales, que estará encabezado por un técnico como director. Este espíritu parece continuarse, si bien a falta de un desarrollo más específico, dentro del *Real Decreto 9/2008*, el cual indica en la versión dada al art. 367.2 del *Reglamento del Dominio Público Hidráulico* que *...el titular deberá disponer de los medios humanos y materiales necesarios para garantizar el cumplimiento de sus obligaciones en materia de seguridad.* Consecuentemente, será necesario requerir a los titulares de las presas y balsas a las que alcance la normativa vigente para que designen –cuando no lo hayan hecho- el equipo y director necesarios en las distintas fases de la vida de cada obra. Esto debería llevar aparejado un protocolo de aceptación o rechazo del nombramiento de los directores, en función de las competencias que ostenten por su titulación profesional. Tal protocolo debe incluir también la autoridad encargada de decidir sobre este particular, así como los efectos del silencio administrativo, en su caso.

   El cumplimiento de las obligaciones relativas a la disposición de equipos humanos encargados de la seguridad y control de las obras por sus titulares durante las fases posteriores a la construcción ha sido motivo de reticencias en algunos casos, alegando el coste económico que ello supone en aprovechamientos particulares, diferentes de los controlados por grandes corporaciones o administraciones públicas. Por ello, y al amparo de lo previsto en el *Reglamento del Dominio Público Hidráulico* sobre la creación de comunidades de usuarios (art. 231), debería estudiarse como una posible solución a tal problema la creación de organizaciones mancomunadas entre los propietarios de presas y balsas para atender a tales deberes, de forma que se repartiesen los gastos derivados de aquéllos. Esta propuesta convendría que fuese, igualmente, objeto de examen por parte de los rectores de los organismos de cuenca y otras autoridades, no sólo por las ventajas que aportaría a los titulares de las obras, sino también para las propias administraciones y servicios responsables de la seguridad, ya que podría facilitar en gran medida la relación con los anteriores, así como la toma de decisiones y la transmisión de instrucciones en casos de emergencia.

i. *Actuaciones sobre obras no legalizables o bajo riesgos inmediatos*.- Con independencia de las medidas de tipo sancionador que proceda tomar contra los titulares de las infraestructuras que se encuentren en situación de ilegalidad, y que deberán seguir los procedimientos ya establecidos para estos casos por la legislación vigente, será necesario que en aquéllas que sean sospechosas de presentar algún riesgo se compruebe tal presunción, así como el alcance de los daños previsibles. Para ello será preciso aplicar, entre otros, procedimientos similares a los que se usen para determinar la clasificación de las obras en función del riesgo potencial, sin que ello implique que se les haya de asignar ninguna categoría de forma oficial, por la incongruencia que esto podría suponer.

   Con tal fin, y partiendo de la información existente en las distintas unidades de las comisarías de aguas -especialmente las encargadas de la vigilancia en campo- convendría seleccionar aquellas obras que aparenten ser más problemáticas, y efectuar un análisis y seguimiento más exhaustivo de las mismas, incluyendo la transmisión de las órdenes oportunas a los titulares, e incluso la ejecución directa por la administración de las acciones que sean precisas, en su caso.

6 NECESIDADES DE APOYOS EXTERNOS PARA ATENDER LAS OBLIGACIONES DE LAS COMISARÍAS DE AGUAS

De todo lo expuesto en el presente documento creemos que se deduce la gran dificultad -por no decir imposibilidad- de que los organismos de cuenca intercomunitarios, y sus comisarías de aguas, en particular, puedan desempeñar en estos momentos, y sólo con los medios habituales, todas las tareas que se han descrito, aun en el caso de que las mismas se repartiesen entre más de uno de sus servicios.

Ello nos lleva a la necesidad de prever la obtención de apoyos externos para tales cometidos, al menos hasta una hipotética consecución de mayores recursos propios. A propósito de esto, la modificación del *Reglamento del Dominio Público Hidráulico* contenida en el *Real Decreto 9/2008* dedica el art. 365 a las denominadas *entidades colaboradoras en materia de control de la seguridad de presas y embalses*, definidas como aquellas organizaciones públicas o privadas que -bajo contrato, y previamente habilitadas mediante el título correspondiente- estarán autorizadas para colaborar con la administración competente en las labores de control, de carácter técnico o especializado, relativas a la seguridad de presas y embalses, las cuales deberán ser las encargadas de prestar dicho apoyo.

En estos momentos están pendientes de crearse los registros en los que figuren las citadas entidades, así como las condiciones y procedimiento para obtener tal calificación, y también sus posibles actividades y competencias, por lo que aún no es posible hacer una planificación detallada de los posibles contratos a celebrar con las mismas, fuera de lo que con carácter general prevén las normas legales vigentes sobre contratación en el sector público. No obstante, se entiende que el apoyo a recabar iría en dos líneas de trabajo fundamentales:

– Actividades de carácter básico, entendiendo por tales las de carácter habitual de seguimiento y control, que correspondería desempeñar según el ritmo de la tramitación de expedientes y cumplimiento de condiciones impuestas por la administración responsable a este tipo de obras. Tienen la dificultad de que no resulta sencillo pronosticar su alcance en un determinado plazo temporal, ni valorar los recursos humanos y materiales que habrían de ser objeto de contratación. Por ello, sería necesario estudiar si existe alguna forma de contratación que tenga suficiente flexibilidad como para adaptarse a unas condiciones de trabajo como las que se describen, que hacen complicado cerrar un pliego con unas mediciones y presupuesto ajustados a las necesidades del caso.
– Actividades de carácter puntual o extraordinario, como podría ser el caso de las que hayan quedado atrasadas y necesiten una puesta al día para un conjunto determinado de obras, más fáciles de precisar y calcular económicamente.

7 CONCLUSIONES

Teniendo en consideración el elevado número de obras hidráulicas existente dentro de las cuencas hidrográficas intercomunitarias españolas, la disparidad de las situaciones que se dan en las mismas, tanto desde el punto de vista técnico, como del administrativo, así como el incremento de los riesgos que se derivará del progresivo envejecimiento de las mencionadas infraestructuras, se considera necesario iniciar a la mayor brevedad posible un período de reflexión y planificación sobre las líneas de trabajo a seguir en el futuro por las diferentes administraciones públicas implicadas en el control y la seguridad de las presas, embalses y balsas. Dentro del mismo, y sin ánimo restrictivo, se deberían acometer las siguientes acciones:

a. Esclarecer de forma precisa con los servicios centrales del Ministerio de Medio Ambiente, y Medio Rural y Marino, y los organismos de cuenca dependientes del mismo, así como con la Dirección General de Protección Civil y Emergencias (Ministerio de Interior), y las administraciones autonómicas afectadas, las competencias que corresponde ejercer a cada unidad, y las obras a las que atañen. Esto debería incluir también la revisión y modificación, cuando se precise, de las disposiciones legales en curso, con el fin de eliminar contradicciones y situaciones dudosas, así como tomar las precauciones adecuadas para que las normas que se publiquen en el futuro, particularmente por las administraciones autonómicas en la asunción de sus nuevas competencias, resulten compatibles con el marco común establecido por la administración estatal.
b. Igualmente debe hacerse con aquellos conceptos ambiguos, incoherencias o indeterminaciones de orden técnico o administrativo que aparezcan en la legislación vigente.
c. Establecer mecanismos y sistemas de coordinación entre las diversas administraciones implicadas en la gestión de las presas, embalses y balsas.
d. Asignar las competencias relativas a la seguridad y vigilancia de presas, embalses y balsas que deban asumir las comisarías de aguas a la unidad o unidades de la misma que se entienda más adecuadas, así como dotarlas -en lo posible- de personal y medios especializados.
e. Preparar una programación coherente -a corto y medio plazo, cuando menos- para el desarrollo de las actividades descritas en el punto 5 de este documento, o de aquéllas que se entiendan necesarias para el cumplimiento de las obligaciones de los organismos de cuenca intercomunitarios.
f. Poner en marcha el desarrollo de las obligaciones más urgentes en materia de seguridad que se encuentren pendientes, particularmente las propuestas de clasificación a que se alude en la disposición transitoria tercera del *Real Decreto 9/2008*, y las que se deriven de la categoría asignada en cada caso.

g. Impulsar, en la medida en que resulte factible, la formación técnica de los funcionarios y empleados al servicio de las comisarías de aguas y demás administraciones que deban atender estos asuntos, incluyendo el adiestramiento en el uso de herramientas informáticas específicas.
h. Valorar la posibilidad y conveniencia de crear un organismo público independiente encargado de la vigilancia y control de las presas y balsas, con fines y atribuciones similares a los que ostenta en España el Consejo de Seguridad Nuclear para las instalaciones nucleares y radiactivas. Esta idea surgió de forma paralela a la de la publicación de una ley sobre seguridad de presas y embalses, aunque nunca llegó a concretarse ninguna de las dos. En todo caso, y aun en el supuesto de que algún día se materializase, no parece probable que ello exima por completo a los organismos de cuenca de sus responsabilidades actuales sobre las presas y balsas.

Ciudad Real, mayo de 2010.

Monitoring, inspection and diagnoses activities

Dam Maintenance and Rehabilitation II – Romeo García et al. (eds)
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Desarrollo de un Protocolo de Auscultación Geológica y estructural, para su aplicación en presas de fábrica antiguas: Presa de La Jocica, Río Dobra, Asturias; Presa de La Tosca, Río Cuervo, Cuenca

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RESUMEN: Más de dos centenares de presas españolas tienen una edad superior a 40 años y no cuentan con infraestructura alguna de auscultación o control de la presa o del cimiento. En agosto de 2009, Navarro Generación S.A., en colaboración con la Fundación Leonardo Torres Quevedo de la Universidad de Cantabria, pone en marcha un proyecto de investigación destinado al desarrollo de un protocolo de auscultación que permita el mínimo control de estructuras y cimentaciones, generalizable para presas antiguas de fábrica.

Se presentan en esta comunicación los resultados obtenidos hasta la fecha en el desarrollo del protocolo mencionado, en lo referente a las cuatro características que se han considerado como controles mínimos y que se pueden llevar a cabo sin las infraestructuras anteriormente citadas, como son el control topográfico de movimientos, el control de deformaciones mediante extensómetros sencillos, el control de filtraciones mediante sistemas de drenaje específicos y el control y evolución de las características geológicas de la cimentación.

La investigación se desarrolla en las presas de La Jocica en el Río Dobra, Asturias, y en la presa de La Tosca en el Río Cuervo, Cuenca, y en sus respectivos entornos geológicos.

ABSTRACT: More than two hundred's spanish dam are older than 40 years and does not have any monitoring infrastructure or control of the dam foundation. In August 2009, Navarro Generación S.A., in collaboration with the Fundación Leonardo Torres Quevedo of the University of Cantabria, starts a research project destined to the development of a monitoring protocol that allows the minimum control of structures and foundations, that can be generalized for other old concrete dam.

Here we show the actual results obtained in the development of the mentioned protocol, regardant the four characteristics that have been considered to be minimum controls and that can be carried out without the infrastructures previously mentioned, like they are the topographic control of movements, the control of stress by means of simple extensometers, the control of filtrations by means of specific systems of drainage and the control and evolution of the geological characteristics of the foundation.

The investigation has been developed in the La Jocica dam, Dobra River, Asturias, and in La Tosca dam, Cuervo River, Cuenca, and in theirs geological environments.

1 INTRODUCCIÓN

1.1 *Las presas*

Conforme con la información recogida en la página web del Ministerio de Medio Ambiente, de las 728 presas existentes en España de más de 15 metros de altura, consideradas grandes presas, 578 son presas de fábrica en sus distintos tipos, gravedad, arco-gravedad, bóveda, contrafuertes o mixtas.

Por otra parte, en el inventario del Comité Nacional Español de Grandes Presas de 1970, se registran como tales 506 presas de las que más de 450, son de fábrica y mayores de 15 metros de altura.

Es preciso llamar la atención sobre que un alto porcentaje de estas estructuras, no cuenta con galerías perimetrales o de reconocimiento en roca, péndulos, drenes verticales y piezómetros o controles de cualquier tipo de las características geológicas y deformacionales de la cimentación.

La presa de La Jocica, Asturias, es una estructura en arco que se apoya en dos estribos de gravedad. Con 87 metros de altura, 66 metros de desarrollo en coronación y solamente 7 metros en el zócalo, web SEPREM, cierra una estrecha garganta en el curso alto del río Dobra, afluente del río Sella. Fig. 1.

Construida entre 1959 y 1963, está cimentada sobre una potente formación de calizas negras laminadas y calizas grises masivas de edad Carbonífero superior.

Al igual que la presa de La Jocica, la presa de La Tosca, Cuenca, es una estructura en arco que se apoya en dos estribos de gravedad. Con 34 metros de altura, 54,50 metros de desarrollo en coronación y solamente unos 12 metros en el zócalo, web SEPREM, que cierra también una estrecha garganta en el curso alto del río Cuervo, afluente del río Guadiela, cuenca del Tajo. Fig. 2.

La presa, cuya construcción finalizó en 1964, está cimentada sobre una formación bien estratificada de calizas y dolomías de edad Jurásico inferior–medio.

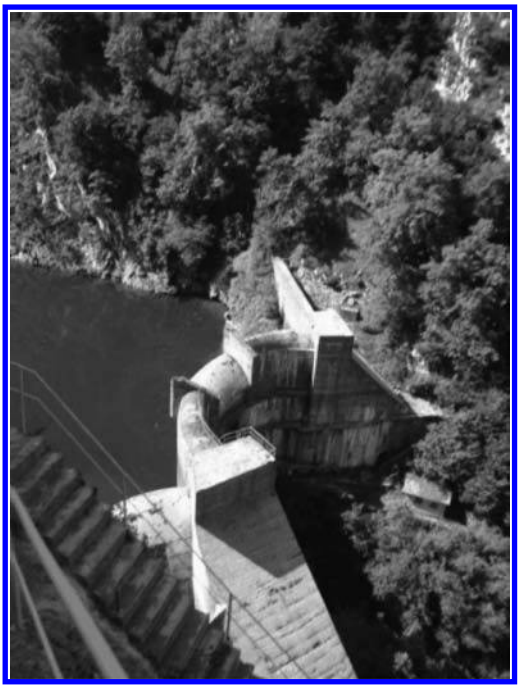

Figura 1. Presa de La Jocica. Río Dobra, Asturias.

Figura 2. Presa de La Tosca. Río Cuervo, Cuenca.

1.2 *Investigaciones realizadas*

En agosto de 2009, se iniciaron las investigaciones en la presa de La Jocica, con el siguiente orden:

a. Reconocimiento geológico general del entorno de la presa.
b. Análisis de las características geológicas y estructurales de ambas márgenes.
c. Análisis de las características geomorfológicas generales del entorno de la presa.
d. Análisis de las características geomorfológicas de las laderas de apoyo de la estructura.
e. Revisión del la distribución y estado de las juntas de hormigonado en la estructura, con especial detalle de las juntas de apoyo del arco central con los estribos de gravedad.
f. Revisión del estado de los drenes instalados y su distribución a lo largo del paramento de aguas abajo de la presa.
g. Revisión de las galerías existentes.

Como consecuencia de estas primeras investigaciones y revisiones realizadas, se llevó a cabo el:

– Diseño y construcción en los talleres de Navarro Generación de un artilugio que se denominó como “extensómetro de alambre”, y a su instalación, en número de nueve, en las juntas de hormigonado principales de la zona del arco central de la presa, como controlador de deformaciones. Fig. 3.
– Construcción in situ de las bases que habrían de sustentar a la Estación Total a utilizar en las medidas topográficas correspondientes al control de movimientos. Fig. 4.
– Limpieza de los drenes existentes con vistas al control de filtraciones. Fig. 5.

Figura 3. "Extensómetro de alambre" para control de deformaciones.

Figura 4. Base y Estación Total en margen izquierda, aguas debajo de la presa.

Figura 5. Dren limpio asociado con una junta de hormigonado.

En la presa de La Jocica, desde el comienzo de las investigaciones en agosto de 2009 y posterior preparación de la infraestructura necesaria ya mencionada, hasta mayo de 2010, se han realizado:

- 7 mediciones deformacionales. Extensómetros.
- 8 mediciones de movimientos. Topografía.
- 7 observaciones de filtraciones. Drenes de la estructura y surgencias del macizo rocoso.

A finales de noviembre de 2009, se realiza la primera visita a la presa de La Tosca. El largo invierno, retrasó el comienzo de las investigaciones y de las instalaciones de control necesarias para llevar a cabo la investigación, construcción de bases topográficas, colocación de los extensómetros y limpieza de drenes, hasta primeros de marzo de 2010, por lo que en el momento actual, mayo 2010, se cuenta con muy poca información. En cualquier caso, se están llevando a cabo las investigaciones con las mismas pautas que en la presa de La Jocica.

2 GEOLOGÍA

El reconocimiento geológico general del entorno de la presa, aguas arriba y aguas abajo, permite detectar circunstancias geológicas que habrán de ser analizadas con detalle en las fases de investigación sucesivas.

Considerando la estabilidad del cimiento, de los estribos y de las laderas, como la característica geológica principal para la seguridad de la estructura, las investigaciones geológicas se basan en la caracterización estructural del macizo rocoso y las determinaciones de la potencial inestabilidad del mismo, así como de las particulares circunstancias geomorfológicas de las laderas para, a partir del análisis de las formas del relieve, detectar o prever las formas de inestabilidad de las formaciones superficiales que recubren las mismas.

En cuanto a la caracterización estructural, en la presa de La Jocica, se llevó a cabo un exhaustivo análisis de la disposición geométrica y características geomecánicas de las discontinuidades estructurales del macizo rocoso. Como resumen de las mediciones, se representa en la Fig. 6, la distribución de discontinuidades en forma de diagrama de rosa.

Durante el desarrollo del estudio geomorfológico general, se pudo observar la presencia de fenómenos de creep o reptación superficial generalizado a lo largo de toda la ladera de margen izquierda.

Dicha reptación, viene definida por la existencia de irregularidades en el terreno, tales como formación de pequeños abombamientos visibles en zonas escarpadas y sin vegetación, así como en la curvatura que toman en su base algunos árboles.

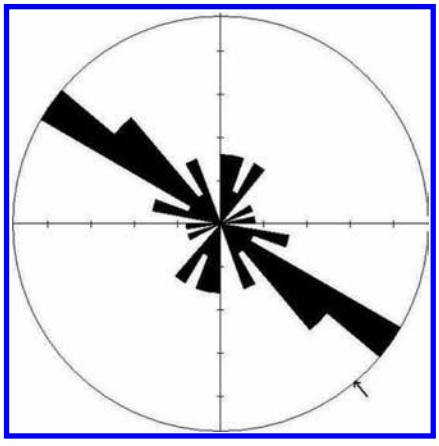

Figura 6. Diagrama de rosa donde se observa la dispersión de la fracturación frente a la homogeneidad de la estratificación, representada por los pétalos de mayor longitud.

Como es bien conocido, la reptación superficial o creep, es un movimiento lento y continuo que afecta a la capa más superficial de la ladera, y puede producirse de varios modos: debido a movimientos de expansión por hidratación o de retracción por deshidratación, por acción de la gravedad o por movimientos en masa debido al deshielo en algunas zonas periglaciares.

Como interpretación general, de la combinación de los estudios de la geología estructural y de la geomorfología de la ladera de margen izquierda, podemos decir que el valle del Río Toneyu, que desemboca en el río Dobra, es, sin duda, un valle glaciar.

Un valle glaciar, se forma a partir de un valle fluvial que es ocupado por el hielo, de modo que los perfiles transversales originales en V típica de valle fluvial, pasa a tener una morfología en U de valle glaciar que proporciona una máxima eficacia para el transporte del hielo. En algunos casos, los perfiles transversales en U suelen tener una cierta asimetría, con una ladera más pendiente que la otra, y en otros casos, adoptan un perfil parabólico, como en nuestro caso.

En el entorno geomorfológico de la presa de La Jocica, la ladera de margen izquierda, presenta claros signos de haber sido afectada por las tensiones provocadas por el peso de la masa de hielo situada en el valle del Toneyu que además de fracturar la roca heterogéneamente, como hemos visto anteriormente en las proyecciones estereográficas, crea un creep, fácilmente identificable en la superficie de la ladera. Fig. 7.

En consecuencia, la generalizada, intensa y dispersa fracturación superficial que afecta a la ladera de la Margen Izquierda, hubo de tener su origen en el levantamiento y consecuente relajación de tensiones que afectó a la Región Cantábrica después de la última glaciación, Würm tardía, hace unos 9.000 a 10.000 años, momento en el comienzan a desarrollarse con más intensidad, los procesos geomorfológicos de deslizamientos, desprendimientos y creeping que, favorecidos por las grandes pendientes que originó el fuerte encajamiento de la red fluvial, han dado lugar al paisaje actual, y condicionado su evolución.

La ladera de margen derecha, está dominada por la estructura de la formación de las calizas masivas, en donde se evidencia la presencia de una falla subvertical más o menos coincidente con el "tajo" en donde se sitúa el comienzo de las escaleras de acceso. Por otra parte, el tramo inclinado y más largo de las escaleras, parece coincidir con el trazado de un plano de potencial deslizamiento, Fig. 8.

Volviendo a centrarnos en las características de la margen derecha, se observa, mediante la toma de medidas, que la estratificación tiene una dirección de buzamiento de 40/48, muy similar a los datos recogidos en la margen izquierda, por lo

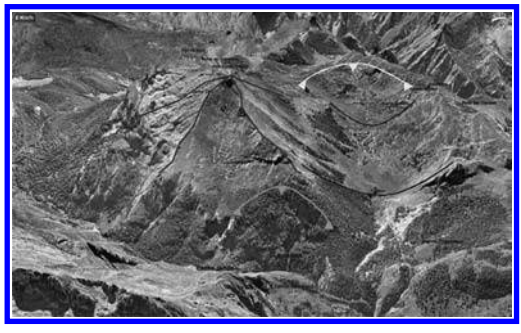

Figura 7. Esquema geomorfológico general de la margen izquierda de la presa y embalse de La Jocica.

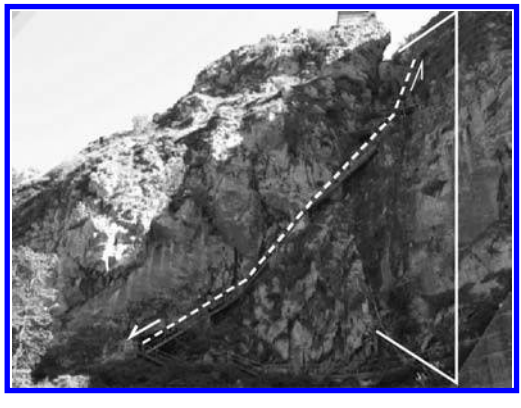

Figura 8. Estructuras mayores afectando a la margen derecha. Falla y plano de deslizamiento.

que podemos descartar la existencia de una gran estructura como podría ser un pliegue.

Esta disposición de los estratos de roca con respecto al talud de la ladera, hace que esta sufra de inestabilidad por vuelco, a diferencia de la margen izquierda en la que predominaban los escarpes y la reptación superficial o creep, propio de una estratificación paralela al talud de la ladera.

3 CONTROL DE MOVIMIENTOS

Como ya se ha comentado, el control de posibles movimientos de la estructura se está llevando a cabo mediante triangulaciones topográficas, basadas en dos bases fijas situadas en ambas márgenes y en la instalación en el centro del arco de un apoyo fijo para la colocación del prisma en cada momento en que se hacen mediciones.

Es preciso llamar la atención, sobre que este tradicional y sencillo procedimiento, permite la interpretación global de posibles movimientos, pues no sólo se puede mover la estructura, la presa, si no que puede haber movimientos o desplazamientos naturales que afecten a los estribos de apoyo. Este caso, arco apoyado sobre dos estribos de gravedad, es generalizable para muchos casos similares existentes en España.

Los resultados obtenidos en las mediciones realizadas desde agosto de 2009 a mayo de 2010, no son especialmente llamativos. Lo que se traduce en que los "movimientos" no han existido o han sido mínimos, entrando las variaciones en el error normal que, hasta las técnicas más modernas en presas de reciente construcción, deben contemplar.

En las Figuras 9 a 14, se muestran los resultados obtenidos en las medidas realizadas en los últimos nueve meses. Las variaciones se manifiestan en milímetros, que deben de incluir el "error" de medición, así como las modificaciones debidas a la temperatura ambiente y de la propia estructura, así como el nivel de embalse en el momento de la medida.

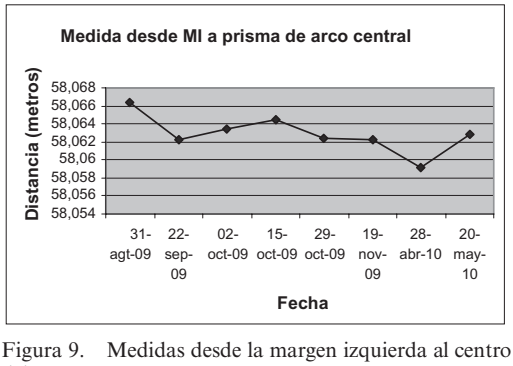

Figura 9. Medidas desde la margen izquierda al centro del arco.

En consecuencia, se considera el procedimiento válido, siempre y cuando se analice e interprete conjuntamente con los demás datos obtenidos en cada campaña de control o auscultación.

Fecha	Distancia m	Altura de agua
31-agt-09	58,0664	23,50
22-sep-09	58,0622	27,40
02-oct-09	58,0634	18,00
15-oct-09	58,0644	21,00
29-oct-09	58,0624	20,80
19-nov-09	58,0622	20,40
28-abr-10	58,0591	26,50
20-may-10	58,0628	28,30

Figura 10. Datos de construcción de la Fig. 9.

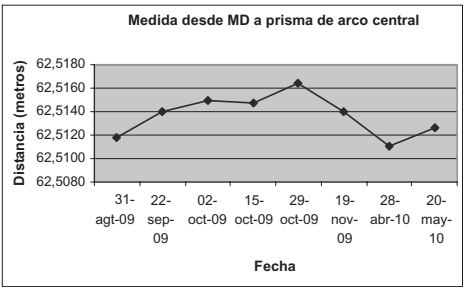

Figura 11. Medidas desde la margen derecha al centro del arco.

Fecha	Distancia m	Altura de agua
31-agt-09	62,5118	23,50
22-sep-09	62,5140	27,40
02-oct-09	62,5150	18,00
15-oct-09	62,5147	21,00
29-oct-09	62,5164	20,80
19-nov-09	62,5140	20,40
28-abr-10	62,5111	26,50
20-may-10	62,5126	28,30

Figura 12. Datos de construcción de la Fig. 11.

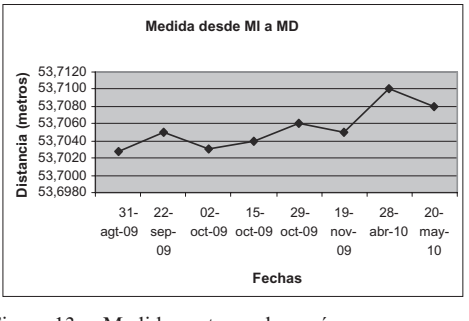

Figura 13. Medidas entre ambas márgenes.

Fecha	Distancia m	Altura de agua
31-agt-09	53,7027	23,50
22-sep-09	53,7050	27,40
02-oct-09	53,7030	18,00
15-oct-09	53,7040	21,00
29-oct-09	53,7060	20,80
19-nov-09	53,7050	20,40
28-abr-10	53,7100	26,50
20-may-10	53,7080	28,30

Figura 14. Datos de construcción de la Fig. 13.

En la presa de La Jocica, se ha tenido en cuenta para el análisis de resultados de las medidas topográficas, como controladoras de posibles movimientos, tanto la altura de agua en el embalse en el momento de la medida, como la temperatura ambiente el día de la medición.

Hasta el momento, el resultado contrastado de estos parámetros no ha sido muy ilustrativo, lo que no quiere decir que no sea importante su vigilancia.

4 CONTROL DE DEFORMACIONES

Como se comenta en la introducción, el control de deformaciones se refiere a la vigilancia de los posibles movimientos relativos a ambos lados de las principales juntas de hormigonado, mediante la medida de las variaciones en cuanto a la abertura o cierre de la junta y de los movimientos de "cizalla".

El "extensómetro de alambre" fabricado para este caso, Fig. 3, consta de muescas grabadas en las placas laterales entre las que se mide las modificaciones en su separación mediante un calibrador. Dado que las deformaciones normales esperadas habrían de ser mínimas, entre cero y dos milímetros, la rotura o despegue del alambre tenso llamará la atención sobre la presencia de una abertura por encima de los niveles normales, y el alambre curvo puede ilustrar sobre la morfología de la deformación en los casos de "cizalla" entre ambos lados de la junta.

En la presa de La Jocica, los resultados obtenidos hasta el momento, mayo de 2010, indican que la deformación por abertura o cierre de las juntas, no ha superado en ningún caso el milímetro, salvo en el extensómetro numero 8 que evidenció una abertura de la junta de 2 milímetros entre febrero y marzo de 2010, abertura que se mantiene en la última medida, mayo 2010. En la Figura 15, se muestra en una tabla los resultados numéricos obtenidos en milímetros, entre los meses de septiembre a diciembre de 2009.

Como continuación de la anterior, en la Figura 16 se reflejan en forma de tabla, los resultados numéricos obtenidos entre los meses de febrero

Ext.	29/sep/09	15/0ct/09	26/Oct/09	18/dic/09
1	140,00	139,00	140,00	140,00
2	140,70	140,00	139,70	140,00
3	141,50	141,00	141,70	140,40
4	141,60	141,00	141,50	140,40
5	140,20	138,00	140,25	139,80
6	139,70	139,00	139,00	139,70
7	142,20	142,00	142,30	140,80
8	142,20	141,00	142,90	142,00
9	142,20	141,00	142,40	142,00

Figura 15. Resultados de las medidas realizadas entre septiembre y diciembre de 2009. Ext: número de extensómetro.

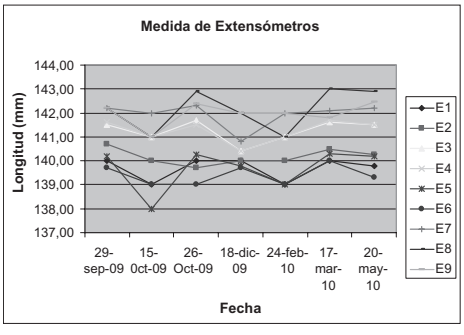

Figura 16. Resultados de las medidas realizadas entre febrero y mayo de 2010.

Ext.	24/feb/10	17/mar/10	20/may/10
1	139,00	140,00	139,80
2	140,00	140,50	140,25
3	141,00	141,60	141,50
4	141,00	141,70	141,50
5	139,00	140,30	140,20
6	139,00	140,00	139,30
7	142,00	142,10	142,20
8	141,00	143,00	142,90
9	142,00	141,80	142,45

Figura 17. Representación gráfica de las variaciones observadas en la distancia entre las muescas de los extensómetros de la primera medida del 29 de septiembre de 2009 y la última realizada en el momento de la elaboración de la presente comunicación, el 20 de mayo de 2010. E: número de extensómetro.

a mayo de 2010. En esta tabla se puede observar la máxima modificación obtenida, de dos milímetros en la abertura de la junta controlada por el extensómetro número 8.

Finalmente, en la Figura 17 se muestran gráficamente las medidas realizadas entre las muescas de los distintos extensómetros y las fechas correspondientes de medición.

Como se ha comentado anteriormente, entre febrero y marzo de 2010 se observó en el

extensómetro número 8, la abertura máxima de 2 milímetros entre las muescas, apertura que se conservó entre marzo y mayo. Fig. 16.

Por otra parte, en abril de este mismo año, se obtuvo la menor distancia medida entre las márgenes derecha e izquierda y el prisma del arco central, a la vez que se obtuvo la menor distancia entre ambas márgenes. Figuras 9, 11 y 13.

Estos resultados, que destacan sobre la regularidad de los obtenidos a lo largo de los controles realizados desde agosto de 2009, han coincidido con los momentos de mayor altura de agua en el embalse.

En consecuencia, el método utilizado se considerará válido, siempre y cuando se analicen e interpreten los resultados obtenidos conjuntamente en cada campaña de control o auscultación.

5 SISTEMAS DE DRENAJE

En la presa de La Jocica, una vez realizadas las primeras operaciones de limpieza de los drenes existentes en el paramento de aguas abajo y con especial interés en los relacionados con las juntas de hormigonado en el arco central, Fig. 5, coincidiendo con las campañas de control de movimientos y deformaciones, se llevó a cabo la observación de los caudales surgentes en los drenes.

En el periodo, agosto 2009 a mayo 2010, en ningún caso se han observado caudales de filtración susceptibles de ser cuantificados en litros por minuto o centímetros cúbicos por segundo.

Como resultado llamativo y que confirma la necesidad del análisis conjunto de todos los parámetros que se han descrito y se obtienen en cada campaña de control o auscultación, las dos últimas observaciones manifestaron una cierta relación entre los niveles de embalse y el comportamiento del sistema de drenaje.

De esta manera, en la revisión del 17 de marzo de 2010 con una altura de agua de 19,60 metros, todos los drenes estaban secos. En la misma observación del 20 de mayo de 2010, con una altura de agua de 28,30 metros, los drenes se presentaban "húmedos", "con gota", "con chorro" o "seco", aunque en ningún caso con un caudal medible en unidades representativas de filtraciones importantes. A pesar de estos resultados que se pueden considerar positivos en cuanto a la ausencia de filtraciones importantes, no por ello deben dejar de realizarse las observaciones pertinentes.

6 PROTOCOLO DE AUSCULTACIÓN

Es indudable que el acceso a la presa de La Jocica no es fácil, y los trabajos en la misma están muy condicionados por las condiciones climáticas de cada momento. Todas las mediciones, topográficas, extensiométricas, drenajes y geológicas, han sufrido esta circunstancia. Por otra parte, los normales retrasos en la instalación de los extensómetros y la limpieza de drenes, no han permitido realizar más observaciones.

Sin embargo, el esfuerzo realizado por todo el personal de Navarro Generación del Centro Dobra en el corto periodo de tiempo en el que se han llevado a cabo las investigaciones que se presentan en esta comunicación, han permitido diseñar un Protocolo de Auscultación Geológica y Geotécnica que, en desarrollo en el caso de La Jocica y de la Tosca, es aplicable a casos similares.

Como conclusión de las investigaciones llevadas a cabo en las presas mencionadas y su entorno, se describen a continuación las premisas o bases en las que se debe de fundamentar el Protocolo de Auscultación Geológica y Geotécnica que permita el control sencillo de los parámetros descritos.

6.1 *Bases del protocolo*

A. Geología

Ha de basarse en la interpretación estructural y geomorfológica del entorno inmediato, aguas abajo y aguas arriba de la cerrada de embalse. Las laderas y la propia cimentación han de ser los ejes de la investigación, con la finalidad de analizar y prever inestabilidades generales y/o locales.

Se recomienda el control de inestabilidades mediante revisión visual en campo de las zonas de riesgo, al menos dos veces al mes durante un periodo anual, cuatro estaciones.

B. Mediciones topográficas. Control de movimientos

El triángulo entre dos bases topográficas en ambas laderas y un punto central en la fábrica es el mínimo requerido para el control de posibles movimientos del conjunto. No obstante, se recomienda el establecimiento de, al menos, dos puntos más de referencia en los estribos de la presa.

Se recomienda el control topográfico de movimientos, al menos dos veces al mes durante un periodo anual, cuatro estaciones.

C. Medida de deformaciones. Extensómetros

Se han de situar en todas las juntas verticales entre tongadas de hormigonado, además de en todas aquellas juntas verticales susceptibles de sufrir deformaciones por extensión o retracción.

Se recomienda el control de deformaciones en las juntas de hormigonado, al menos dos veces al mes durante un periodo anual, cuatro estaciones.

D. Drenes

Ha de haber un dren situado en la junta bajo cada uno de los extensómetros, independientemente de los que las propias características de la fábrica así los pida.

Se recomienda el control de filtraciones bajo los extensómetros y en la las juntas de hormigonado y lugares que así lo exija la fábrica, al menos dos veces al mes durante un periodo anual, cuatro estaciones.

E. Galerías

De existir galería en roca, deberán controlarse las filtraciones mediante drenes y observación visual, así como las posibles deformaciones mediante extensómetros en las discontinuidades de riesgo.

En las zonas de galería hormigonada, el control de filtraciones y deformaciones deberá hacerse en clave y en las juntas de hormigonado en hastiales.

Se recomienda el control de filtraciones bajo los extensómetros y en la las juntas de hormigonado y lugares que así lo exija la fábrica y el fondo de galería en roca, al menos dos veces al mes durante un periodo anual, cuatro estaciones.

F. Aguas de filtración

Es recomendable el análisis de las aguas filtración recogida de los drenes antes mencionados, así como en las zonas de fondo de galería. Como en los controles anteriores, al menos dos veces al mes durante un periodo anual, cuatro estaciones.

G. Informes

Conforme con el Protocolo descrito, se recomienda la recogida de los resultados obtenidos, al menos dos veces al mes durante un periodo anual, cuatro estaciones, en los Informes correspondientes, que deberán ser incluidos en el Archivo Técnico de la Presa y reflejados en el Libro de la Presa.

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Gestión del mantenimiento y la auscultación de las presas del Canal de Isabel II y su relación con las Normas de Explotación y los Planes de Emergencia

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ABSTRACT: Over the past five years, the maintenance management, auscultation, monitoring and inspection dams of Canal Isabel II has been a significant change due to two main causes: the adaptation process to new technologies by deleting the paper and the influence of the Standards of Operation and Emergency Plans for Dams in the organization of such plans. Technological change has been a factor that has contributed to a rethinking of the objectives and methods used. The aim is to exploit all the advantages of the use of information and computer without losing sight of the importance of visual review, including by establishing a systematic approach to the same common in all dams. Plans and Standards dams have served to align and unify the procedure so they can be considered as a support and reference for defining the breadth and depth of the process. As a result, has gone from a situation that needed a series of operations of transcription and physical transfer of paper to another is known in the state and responses to dams in near real time and with much lower error.

1 INTRODUCCIÓN

El Canal de Isabel II (CYII) es una empresa Pública dependiente de la Comunidad Autónoma de Madrid. Se creó en 1851 con el objetivo de suministrar agua a Madrid y desde 1984 extendió su actividad a distribuir al resto de los núcleos urbanos de la Comunidad. En la actualidad es la empresa encargada de gestionar el ciclo integral del agua en la Comunidad Autónoma de Madrid: captación, embalse, aducción, tratamiento, distribución, saneamiento y depuración para más de seis millones de personas, lo que supone el 98% de la población de la Comunidad y uno de cada siete españoles. Para cumplir con esta misión el Canal de Isabel II es titular o explotador de 15 grandes presas y 6 azudes de menor entidad, cuya capacidad total es de 943,5 hm^3.

En la presente comunicación se pretende describir el mantenimiento, conservación e inspección que realiza el CYII en la gestión del conjunto de presas, y su integración con la normativa actual y con los Planes de Emergencia y Normas de Explotación de las presas.

2 CARACTERÍSTICAS DE LAS PRESAS

Para una mejor compresión del mantenimiento de las presas del CYII es necesario destacar las circunstancias puntuales de las mismas que hay que tener en cuenta para su gestión y explotación.

2.1 *Distribución geográfica*

El CYII presta sus servicios en un área vinculada geográficamente a la Comunidad Autónoma de Madrid. Para su abastecimiento capta agua superficial de las cuencas de los ríos Sorbe, Jarama y sus afluentes Lozoya y Guadalix, Manzanares y su afluente Samburiel, Guadarrama y sus afluentes Navalmedio, La Jarosa y Aulencia, y el río Cofio. Capta también agua del río Alberche, aunque en éste solamente gestiona la impulsión y conducción, no la captación. Se trata pues de cuencas relativamente pequeñas y lindantes unas con otras, lo que provoca una cierta concentración de las presas.

Dos de ellas están ubicadas en la provincia de Guadalajara, una en la provincia de Ávila y el resto están ubicadas en la provincia de Madrid. Si bien en distancia esta circunstancia no es muy importante, tiene cierta relevancia en cuanto a trámites administrativos, permisos, etc. debido a las diferentes administraciones afectadas en la gestión.

La gestión está dirigida desde Madrid, que está situado aproximadamente en el punto central de esta zona, siendo los tiempos de desplazamiento desde allí hasta la presa más alejada de una hora y media aproximadamente. Esta circunstancia de cercanía física favorece el mantenimiento de una

relación ágil y cercana entre las personas y las presas y entre las personas entre sí.

2.2 *Variedad en edad de las presas*

El origen del CYII, a mediados del S. XIX, está unido directamente a la construcción de la presa del Pontón de la Oliva, que desde entonces se mantiene en explotación y constituye la presa más antigua de todas las gestionadas; si bien actualmente no se utiliza para la captación y almacenamiento de agua y se mantiene permanentemente vacía, el CYII mantiene gran esmero en su cuidado y conservación y, por supuesto, mantiene en vigor toda su documentación: clasificación, Normas Explotación, Plan de Emergencia y su implantación, etc.

Inmediatamente posterior es la presa de El Villar (1869) que el CYII construyó y que supuso quizá la primera presa de gravedad con la concepción actual; en su momento fue la más alta de Europa. Esta presa permanece en servicio actualmente sin ninguna limitación en su explotación.

Desde entonces se han ido incorporando más presas a la gestión del CYII hasta completar el número actual. Entre ellas se recoge gran variedad de circunstancias que han condicionado su diseño, construcción y explotación y, por tanto, sus características. Algunas de ellas sufrieron la paralización de su construcción por causa de la guerra civil, con reanudación posterior incluyendo cambios en su concepción y diseño y con restricciones importantes en medios materiales y recursos.

A lo largo de los años 50, 60 y principios de los 70 fueron construidas gran parte de las presas actuales, ya con una mayor disposición de medios y técnica consiguiendo calidades mejores.

Entre las presas gestionadas por el CYII se cuenta con prácticamente todas las tipologías de presa al existir 8 de gravedad de planta recta, 4 de gravedad de planta curva, 2 bóvedas y 2 de materiales sueltos (escollera con pantalla asfáltica).

La presa más moderna con que cuenta el CYII es La Aceña, cuya construcción finalizó en 1989 y se incorporó a su gestión en 2003.

La utilización de criterios homogéneos en el mantenimiento y la explotación de todo el conjunto de presas, con edades y circunstancias tan dispares, supone un gran esfuerzo para el titular. Por otra parte, la necesaria adecuación continua a las distintas normativas que van apareciendo obliga a unas inversiones de adaptación muy importantes en relación al coste total de su explotación y mantenimiento.

3 ORGANIZACIÓN DEL PERSONAL

Para llevar a cabo todos los trabajos y actividades necesarios para alcanzar los objetivos de la gestión, el CYII ha diseñado un equipo técnico de explotación de presas, formado por un Director de Explotación y por los 4 equipos de personal siguientes:

- Equipo de operación, vigilancia y mantenimiento. En líneas generales se corresponde con la División de Explotación que incluye, como personal técnico, el jefe de la división y otros tres ingenieros ayudantes.
- Equipo de análisis e interpretación de los datos de auscultación. Se corresponde con la División de Seguridad que cuenta con el jefe de la división y otro ingeniero ayudante.
- Equipo responsable de telecontrol de las maniobras.
- Equipo de mantenimiento de comunicaciones.

Tal y como exige la normativa vigente la organización gira en torno a la figura del Ingeniero Responsable del Equipo Técnico de Explotación, que es la persona al cargo de la seguridad de las infraestructuras.

En el organigrama del CYII, el Ingeniero Responsable es el Jefe de Departamento de Presas.

Los dos primeros equipos dependen directamente del Director de Explotación ya que todas sus funciones y atribuciones son específicas de la explotación de presas. En cambio, los otros no dependen jerárquicamente del Jefe de Departamento de Presas. Ambos son servicios generales para toda la empresa y cada uno desempeña labores específicas dentro de ella, entre las que la atención a las presas constituye una parcela singular.

Existen unos protocolos de actuación y unas prioridades en caso de necesidad o emergencia, para que una parte importante de estos equipos estén a disposición del Director de Explotación en caso de necesidad.

4 REQUISITOS NORMATIVOS

Las dos normativas actuales aplicables a las presas del CYII son: la Instrucción para el Proyecto, Construcción y Explotación de Grandes Presas de 1967 y el Reglamento Técnico sobre Seguridad de Presas y Embalses de 1995.

Ambas normativas recogen el requisito de que el titular de una presa defina un plan de mantenimiento e inspecciones de la presa. Incluso el Reglamento señala en su Art. 33.2 *"El Director de Explotación redactará un informe anual recogiendo los resultados de las inspecciones realizadas y de la auscultación …"* que, en el caso de las presas de categoría A, deberá ser enviado a la Administración competente.

Además la legislación vigente obliga al titular de la presa a elaborar e implantar una serie de z

normativos como el Plan de Emergencia o las Normas de Explotación, que están estrechamente relacionados, tanto con el Plan de Mantenimiento e Inspección como con el Plan de Auscultación. De hecho, ambos planes deben formar parte del documento de Normas de Explotación.

5 ELABORACIÓN DEL PLAN DE MANTENIMIENTO E INSPECCIÓN

5.1 *Criterios en la elaboración*

Los criterios seguidos en la elaboración y redacción del Plan de Mantenimiento han sido los siguientes:

- Requisitos Normativos: cumplir con lo recogido en la Instrucción y Reglamento.
- Requisitos de Normas de Explotación y Planes de Emergencia: se debe inspeccionar y realizar un seguimiento de todas las hipotéticas causas o signos que evidencien una anomalía en la estructura o explotación de la presa.

  En la elaboración de los Planes de Emergencia se realizó un estudio de las causas que podían motivar la declaración de cada escenario por diferentes motivos, definiendo para los mismos una serie de umbrales.

  En muchas ocasiones los umbrales asociados a inspecciones son cualitativos y es preciso definir las tareas de cómo y cuándo hay que realizar las observaciones pertinentes para determinar si hay señales de riesgo.
- Experiencia del Canal de Isabel II: el CYII es una empresa con más de 150 años de experiencia en la explotación de infraestructuras hidráulicas, factor que ha sido fundamental en la definición de las tareas de mantenimiento de todos los órganos de las presas para definir los trabajos a realizar y su periodicidad.
- Interconexión del trabajo diario con los requisitos normativos.
- Correcto flujo de la información recopilada: por la cantidad de presas gestionadas, el volumen de datos de inspecciones y mantenimiento es enorme. Es necesario disponer de un sistema que favorezca el flujo de la información que cada puesto y situación requiere en cada momento.
- Plan dinámico: El plan de mantenimiento debe ser actualizable en el tiempo, recogiendo las distintas circunstancias de la explotación, cambios en la normativa, gestión, instalaciones, etc. Ello se debería reflejar a su vez en los requerimientos de los documentos normativos (Normas de Explotación y Plan de Emergencia) de forma ágil, evitando así ese desfase que provocaría la obsolescencia rápida de los documentos.

5.2 *Fases en la elaboración*

Las fases que se han seguido en la elaboración del Plan de Mantenimiento han sido:

- Inventario de Instalaciones: el primer paso fue elaborar un listado con de todas las instalaciones susceptibles de mantener e inspeccionar. El inventario de las instalaciones se ha realizado en cuatros niveles diferentes:
  - Unidad (ejemplo: Presa de Pinilla).
  - Sistema (ejemplo: Aliviadero).
  - Equipo (ejemplo: vano izquierdo).
  - Componente (ejemplo: motor eléctrico).
- Inventario de aspectos a inspeccionar y/ó mantener (Definición de tareas). A partir los requisitos impuestos por la normativa y la experiencia del CYII se establecen el listado de tareas que se debe ejecutar para asegurar la inspección de todos los aspectos críticos definidos en los Planes de Emergencia y Normas de Explotación.
- Definición contenido de cada tarea.
- Inventario y gestión de recursos.
- Establecimiento de periodicidades.

6 FUNCIONALIDAD DEL PLAN DE MANTENIMIENTO

Como ya se ha comentado, para poder gestionar toda la información y registro que se generan al realizar las inspecciones y mantenimiento de las presas ha sido imprescindible el desarrollo de una herramienta informática en red capaz de tratar el flujo de la información y como ayuda a la toma de decisiones. La herramienta informática desarrollada ha sido la aplicación GOMA (Gestión de la Operación, Mantenimiento y Auscultación de Presas).

Esta aplicación es un programa de diseño propio que fue concebido a partir a las exigencias particulares de las presas del CYII.

Todas las labores de mantenimiento e inspección las realizan dos equipos diferenciados (un equipo de mantenimiento y otro equipo de operación).

La unidad de operación tiene asignadas las siguientes labores:

– Captura de datos de auscultación.
– Limpieza y conservación de la obra civil.
– Operación de órganos de desagüe y toma.
– Inspecciones de los paramentos, galerías, contactos presa-cimiento.

Es el personal propio de la presa, los que conocen cada rincón y advierten cualquier cambio o incidencia en sus galerías, paramentos, etc. Para este cometido el CYII destina, en turno de mañana, dos oficiales por cada dos presas, todos los días de la semana, con la singularidad de la presa de El Atazar que cuenta con personal las 24 horas y 365 días al año, organizado en tres turnos de trabajo.

La unidad de mantenimiento es un equipo especializado en la conservación de los equipos mecánicos y eléctricos de las presas. Es un equipo itinerante que cada semana se dedica a una presa para comprobar el buen estado de todos los elementos de maniobra y de la instalación eléctrica, incluyendo grupos electrógenos, transformadores, etc. Este grupo de mantenimiento está formado por 10 oficiales y un titulado medio para todas las presas del CYII.

Cuando los oficiales de ambas unidades realizan cualquier tarea de mantenimiento o inspección rellenan los datos exigidos en GOMA y, si detectan cualquier anomalía o incidencia, la deben detallar en el apartado de "observaciones".

La aplicación envía automáticamente las observaciones generadas el día anterior al capataz correspondiente que, a la vista de lo expuesto en la orden de trabajo, puede generar una incidencia.

Las incidencias se envían a los Ayudantes de División para su análisis y generación de propuesta de acción correctora, si procede.

La aprobación de la acción correctora corresponde al Jefe de la División de Explotación.

Todas estas acciones y mensajes los ejecuta la aplicación automáticamente, archivando todos los registros, esto permite obtener un histórico del mantenimiento correctivo ejecutado en todas las instalaciones.

Para que la aplicación obtenga los resultados deseados ha sido necesario dotar de ordenadores portátiles a las brigadas de trabajo y dotar a las salas de explotación de presa (salas de emergencia) de una red de comunicaciones suficiente para la descarga de los datos de explotación. En total se dispone de 20 ordenadores portátiles.

7 REVISIÓN DEL PLAN DE MANTENIMIENTO

Una condición indispensable para el correcto funcionamiento del Plan es su propio mantenimiento, entendiendo por tal, los procesos de actualización y revisión.

Cada año, y tal como recoge la normativa para las presas clasificadas como tipo A, se redacta un informe anual del mantenimiento e inspección de cada presa donde deben figurar todas las tareas que se han realizado en el último año reseñando las anomalías e incidencias detectadas así como las medidas correctoras que se han tomado.

Con una periodicidad mayor se deben acometer los procesos de revisión de los documentos normativos (Normas de Explotación y Plan de Emergencia), lo que vuelve a significar una revisión del Plan de Mantenimiento, ya que éste forma parte de las propias Normas.

Y por último, en las Revisiones de Seguridad y Análisis de la Seguridad de las presas, uno de los aspectos fundamentales a analizar es el estado general de las instalaciones determinando si el mantenimiento e inspección proyectados es adecuado.

Los tres procesos anteriores suponen una revisión del Plan de Mantenimiento, de la que se puede desprender:

– Modificaciones en el inventario.
– Añadir o eliminar tareas.
– Modificación del contenido de alguna tarea.
– Modificar las periodicidades.

La aplicación GOMA se ha diseñado de forma para que cualquier modificación del Plan se pueda realizar fácilmente y quede constancia de la misma.

8 ELABORACIÓN DEL PLAN DE AUSCULTACIÓN Y VIGILANCIA

La sistematización efectuada en estos apartados se inicia con el documento "Auscultación y vigilancia de las presas" editado en 1992. Se estructuró un plan de auscultación en cuatro fases: toma de datos, procesado, análisis e interpretación y emisión de informes. Esta sistemática se reforzó con la adaptación en 1999 del Canal de Isabel II a la Norma ISO-9002.

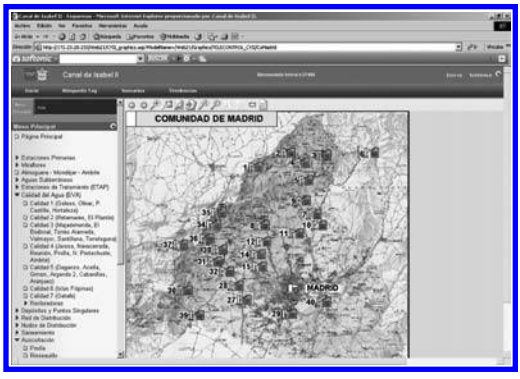
De acuerdo con los postulados anteriores se establecieron dos categorías de instrumentos: instrumentos detectores e instrumentos de apoyo.

Los instrumentos detectores se caracterizan por:

- Tener un número reducido en cada presa.
- Ser leídos con una frecuencia diaria.
- Disponer de umbrales de alerta y de alarma.

Su misión es detectar inmediatamente toda desviación respecto al estado normal de la presa y avisar al explotador sobre la necesidad de iniciar rápidamente las medidas de control y de verificación adecuadas y, a continuación, tomar las decisiones de explotación convenientes.

Los instrumentos de apoyo son aquellos cuyo fin es ayudar a la interpretación de todo fenómeno anormal que pueda presentarse durante la vida de la obra. La frecuencia de sus lecturas es variada (diaria en algún caso, semanal, mensual o anual como las mediciones topográficas). Cuando se produce un acontecimiento anormal o simplemente excepcional, la frecuencia de lectura de estos instrumentos se aumenta sensiblemente.



9 APLICACIONES INFORMÁTICAS

9.1 *GOMA*

Como se ha comentado, la aplicación informática GOMA también es el principal soporte de registro y gestión de los datos de auscultación.

Cada una de las presas, tiene asociadas diferentes tareas de recogida de datos con periodicidades diarias, semanales y mensuales. En las tareas diarias, los datos registrados se almacenan para su posterior tratamiento y en el caso de los aparatos detectores se analiza el dato real con el valor umbral asociado a la situación actual de la presa.

9.2 *CPCillo*

Además de la toma de datos "in situ" efectuada de acuerdo con las tareas de GOMA, los datos de auscultación son transmitidos mediante comunicación Ethernet a la estación maestra concentradora ubicada en el CPC (Centro Principal de Control), junto con los correspondientes a otras magnitudes (niveles en embalse o en canal, apertura de válvulas, etc.) procedentes de sensores ajenos a la auscultación.

Para facilitar el conocimiento de los valores de la señal, el propio sistema de telecontrol dispone de programas para visualización instantánea de la medida y de su evolución en el tiempo, pudiendo seleccionarse directamente el periodo de análisis.

Este programa de visualización en tiempo cuasi-real fue concebido para operar desde el CPC. También puede ser activado desde los ordenadores personales del equipo técnico pues se ha establecido una conexión mediante protocolo TCP/IP con protección por contraseña que permite el acceso directo al banco de datos del CPC, de ahí que con el tiempo, se haya adoptado la denominación de CPCillo. Esto permite un mejor control y la posibilidad de una rápida respuesta, lo que implica un notable incremento de la seguridad.

Cuando la medida de auscultación se recibe en el CPC, sigue un proceso de forma automática formado por los siguientes pasos:

- Filtrado de la señal (sólo se acepta un cambio cuando el porcentaje de cambio supera un valor prefijado). Su objetivo es eliminar los cambios muy pequeños en los valores debidos al "ruido" en la transmisión.
- Alisado (para eliminación de datos instantáneos espurios).
- Comprobación de límites (los datos se comparan con valores umbral de alerta y alarma fijados previamente).
- Comprobación de gradiente (la velocidad de cambio de valores se compara con un porcentaje límite de cambio sobre el rango de la medida fijado previamente).
- Archivo del dato.

Existen cuatro valores umbral para la señal: dos inferiores (alerta y alarma) y dos superiores. La superación de estos valores o la superación del gradiente fijado generan un aviso que deberá ser reconocido de modo manual por el personal del CPC.

Para estos casos se han establecido protocolos de actuación ante situaciones anómalas. En relación con este tipo de análisis se ha definido el concepto de "situación anómala" como aquella que se produce cuando, al tomar un dato de auscultación, se observa un valor que sobrepasa los valores umbral y queda fuera del intervalo de aceptación.

10 CONTROL. DE UMBRALES DE EXPLOTACIÓN

10.1 *Avisos y alarmas*

Los umbrales recogidos de las Normas de Explotación y en los planes de emergencia de presas se programan en la aplicación GOMA para ser comparados con los datos diarios de auscultación.

La aplicación avisa de dos formas posibles: Límite Superior y Límite Inferior, en caso de la superación del valor umbral y a través de dos vías de comunicación distintas: correo electrónico y SMS.

Los límites establecidos se clasifican en función de la importancia que tienen. Se establece un LS1 y LI1 como preaviso de una posible situación anómala y un LS2 y LI2 cuando el valor sigue aumentando y se aleja aún más de la situación de normalidad.

Todos los días sobre las 12:00 a.m. el personal técnico del Departamento de Presas recibe tanto el correo electrónico como el SMS informativo del estado estructural.

10.2 *Cambio de umbrales*

Con la experiencia de los años en la gestión de la auscultación de las presas del Canal de Isabel II y el conocimiento del comportamiento de las mismas, se han presentado varias situaciones en las que de forma evidente se ha llevado a cabo un cambio en los valores umbral de aviso. Entre ellas figuran:

- Obras de limpieza de drenes que conllevan un incremento significativo de las filtraciones en la presa.
- Aumento repentino de filtraciones en los aforos zonales como consecuencia de la importante influencia de la nieve acumulada en días previos.
- Periodos de bajas temperaturas que ocasionan aperturas de juntas de poca importancia.

Hay que decir que los umbrales adoptados de las NEX y PE fueron deducidos a partir de los datos estadísticos que se disponían hasta ese momento. En muchos casos, esa serie temporal de datos, no representa correctamente la realidad de las variables externas asociadas a la presa.

Por este motivo durante los últimos dos años, la División de Seguridad de Presas ha realizado cambios en los valores umbral, justificados a partir de alguna de estas causas.

11 INFORMES DE AUSCULTACIÓN

La normativa actual señala la obligativa legal de realizar un informe anual de comportamiento por parte del titular de la presa en el que se analizan los datos de auscultación tomados en el último año.

El Canal de Isabel II, lleva acumulados más de veinte años de experiencia en este aspecto, redactando un informe anual de comportamiento para cada una de las presas, además de informes mensuales y trimestrales en el caso de la presa de El Atazar.

La evolución de su estructura y contenido ha estado fuertemente influenciada tanto por las Normas de Explotación y los Planes de Emergencia de Presas como por la propia normativa de calidad implementada por el CYII.

12 CONCLUSIONES

Debido a la adaptación del proceso a las Normas de Explotación y los Planes de Emergencia de Presas así como al empleo de las nuevas tecnologías con abandono del soporte papel, la gestión del mantenimiento, conservación e inspección en las presas del Canal de Isabel II ha registrado un importante cambio a lo largo de los últimos cinco años.

Los Planes y Normas de las presas han servido para ajustar el procedimiento y unificar criterios de modo que pueden considerarse como un elemento de apoyo y referencia a la hora de definir la amplitud y profundidad del proceso.

El cambio tecnológico ha sido un factor que ha contribuido a un replanteamiento de los objetivos y métodos utilizados. El objetivo es aprovechar todas las ventajas que supone el uso de la informática y telemática sin perder de vista la importancia de la revisión visual, incluso estableciendo una sistemática para la misma común en todas las presas.

Como resultado, se ha pasado de una situación que precisaba realizar una serie de operaciones de transcripción y traslado físico de papeles a otra en la que se conoce el estado y respuesta de las presas prácticamente en tiempo real. Además el concepto de calidad ha entrado a formar parte de las prioridades en las diferentes actividades.

En todo el proceso de definición de los planes de Mantenimiento y Auscultación se ha intentado que todos los factores (recursos materiales y humanos) que intervienen en la explotación y gestión de las presas estén relacionados entre sí.

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Assessment of dams and dikes behavior with a fibre optics based monitoring solution

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ABSTRACT: The optical fiber based monitoring solution can detect and localize the early signs of malfunctioning, such as leaks or instability, and enhance the long term performance of the dams. It is based on the combination of a technical textile, embedded fiber optic sensors, a related instrumentation, an appropriate design and installation of the system, and a deep data analysis. The system is able to monitor several tens of kilometers with distributed and continuous measurements, which increases the accuracy and the speed of response, crucial parameters to prevent collapse. Detection performance as low as 0.1 l/min/m for leakage and as low as 0.02% soil strain has been validated on experimental works scale 1:1. This solution is used on several dikes and levees.

1 INTRODUCTION

Monitoring the behavior of new and old dams is a key issue in the field of dam maintenance. Thanks to the early detection of precursors of malfunctioning such as leakage, erosion, relative deformation or settlement, it is possible to react very fast when necessary or to check periodically the slow dam ageing process and to plan the maintenance works in due time.

The TenCate GeoDetect® monitoring technology detects both internal erosion and instability at an early stage. It is based on the combination of a technical textile with embedded fiber optic cables that are connected to appropriate optical instrumentations to measure soil temperatures and strains. The current performance of the GeoDetect® global solution has been validated in France and in The Netherlands: the strain resolution can be as little as 0.02% and the temperature resolution as little as 0.1°C, with spatial resolutions of 1 m up to 20 km in distance.

It has been used in two different test series in the recent Dutch experimental IJkdijk project. In the first test series, based on the simulation of a 100 m long macro-instability dyke, the GeoDetect® solution has been the only system among ten others able to detect and localize the start of the dike slide more than 2 days before its collapse during the 4 full days of experimentation time. In the second test series, based on the simulation of piping, it has been the only system able to generate real-time alarms, 2 days prior to failure. The GeoDetect® monitoring and early warning solution is now being used in several real sites.

After introducing the principle of this monitoring solution and describing its real performance based on the experimental dikes, the installation and the data analysis on several dykes will be described.

2 THE FIBER OPTICS BASED MONITORING SOLUTION

2.1 *The textile fiber optics composite sensor*

The fiber optics have been widely used for many years in civil engineering applications and specially hydraulic works such as concrete and earthdams, levees and dikes. By embedding optical cables into a geotextile fabric (Fig. 1), GeoDetect® is the first system designed specifically for geotechnical and

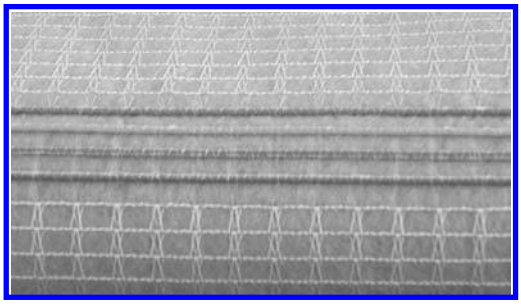

Figure 1. The TenCate GeoDetect® S-BR textile composite sensor embedding 4 optical cable, 2 for temperature and 2 for strain measurement.

hydraulics applications. The geotextile fabric, e.g. a textile installed in the soil, enhances its mechanical and hydraulic properties by in-plane drainage capability, anchoring interface with the soil, soil reinforcement, separation and filtration.

For example the drainage properties of the geotextile combined with the temperature measurement with the optical fibers improves the speed of leakage detection by collecting the water-flow and draining it faster to the optical fiber. Moreover, the filtration properties of the geotextile increase the stability of the soil to prevent the internal erosion process.

Also the high soil-textile interface friction properties and the strong link between the optical cable and the geotextile help to transfer and detect very small soil strains. The GeoDetect® sensor is designed to detect the first steps of internal erosion processes and hydraulic works instability. The detection of the leaks, which is the early stage of the internal erosion process, is assessed through the measurement of temperature changes using the passive method or the active heat pulse method (Radzicki et al., 2009). First stages of dike settlement or sliding are detected by strain measurement. Measuring at the same time and at the same location both temperature for leaks detection and soil strain for soil movement detection increases the probability of obtaining the right precursors of a malfunctioning.

This development was handled through a partnership of companies and public research institutes including several authors of this paper within the European Eureka R&D project "SafeDike".

2.2 *The monitoring and early warning system*

The GeoDetect® monitoring system combines the benefits of geotextile materials with the latest optical sensing and measurement technologies. It uses either the stimulated Brillouin or Raman scattering technology in single mode or multi-mode optical fibers to measure both strain and temperature, or local Fiber Bragg Grating (FBG) sensors placed on single mode fibers to measure strain very precisely both under static or dynamic conditions.

Combined with the appropriate software and instrumentation (Fig. 2), it provides an innovative solution for the multi-functional requirements of geotechnical and hydraulic applications in addition to data capture.

Different monitoring strategies may be designed, for example temporary monitoring, or continuous monitoring to be used as an early warning system.

In comparison to existing detection systems, the GeoDetect® solution is a distributed and continuous measurement along the whole hydraulic works length, which increases the accuracy and the speed of response, both crucial parameters to prevent collapse. It can provide a leak and deformation location with a spatial resolution of 1 meter, or even 0.5 m in some cases. The system is able to monitor several tens of kilometres.

2.3 *The full monitoring solution*

Beside the hardware components of the system, important parts of this global solution are the design and implementation of the system within the work and the interpretation of the raw thermal

Figure 2. The Raman instrumentation for temperature measure ment (on top) and the Brillouin instrumentation for strain measurement (at the bottom).

Figure 3. Installation of one GeoDetect® S-BR sensor strip in the Ijkdijk macro-stability dike.

and strain data. The detection of very small water pipes (or wormholes) for example has been improved recently. The physico-statistical and signal processing methods developed by EDF and used by geophyConsult allow the proposed system to be able to send « early warnings » which is compatible with long-term monitoring. The GeoDetect® solution has been validated on several 1:1 scale experimental works and real dikes in-use, as presented in the next sections.

3 PERFORMANCE EVALUATION OF DAM STABILITY MONITORING

First stages of dike settlement or sliding are detected by strain measurement. An experimental dyke 100 m long and 6 m high was built within the Dutch IJkDijk (Smart Calibration Dike) project to test the ability of sensoring systems to detect dam failure due to internal instability (Fig. 4). The IJkdijk-site is located in the North-East of the Netherlands, close to the German border.

Experiments were carried out with the following objectives. The first goal was to test whether the state-of-the-art multi-sensor networks and IT-tools can supplement standard visual inspections and existing water management systems. About ten different techniques were used. The second objective

Figure 4. Aerial view of the IJkdijk experimental dike during the macro-stability experiment, the upstream basin on the left filled with water, loading containers on top, and the trench diged out 2 m from the dike toe.

Figure 5. Placement of the 4 GeoDetect® sensor strips along the IJkdijk macro-stability dike, at the crest (TOP), at the middle of the slope (MIDDLE), at the dike downstream toe (TOE), and 2 m from the downstream dike toe (FIELD).

was to obtain knowledge about failure mechanisms and to carry out model validation (IJkdijk, 2008).

The fiber optics GeoDetect® system was installed lengthwise to the dike at four different levels: top on the crest, at the middle, at the toe of the downstream slope, and 2 m meters away from the downstream toe (Fig. 5). The four strips, 100 m long each, were all connected together in a loop to the instrumentation in a cabinet at about 50 m from the experimental dike. Both optical cables for strain measurement using the Stimulated Brillouin Scattering and for temperature compensation measurement using the Raman Scattering technologies were embedded onto the textile strips.

In September 2008, during the test, the global factor of safety of the dyke was decreased step by step by digging a trench at the downstream toe and by increasing the internal water pressure till failure by slippage of an area of the downstream face. The failure took place on the third day of the test, during bright sunny conditions. The visible part of the failure took only 40 to 50 seconds, so it was a fast process. The monitoring system worked perfectly as it was able to detect and localize the early signs of soil movements 2.5 days before the dike collapses, i.e. only one day after the start of the experimentation. Very small soil strains of 0.03% on the top strip and less than 0.01% on the middle strip

were measured, see round red circles in Figure 6. The boarders of the instable zone inside the dyke body were clearly localized more than one day before the failure occurred, when nothing was visible from outside (Fig. 7). They exactly correspond to the area of the slipping slope.

Soil strain detection and measurement with the GeoDetect® monitoring solution had already been proven in previous earthworks, such as walls or embankments over instable soils, with another fiber optics technology, the Fiber Bragg Gratings (FBG) (Nancey et al., 2006; Briançon et al., 2006). This IJkdijk macrostability experimentation, confirmed the capability of this system to detect and localize small soil strains in hydraulics works on longer distances by using the Brillouin distributed scattering technology.

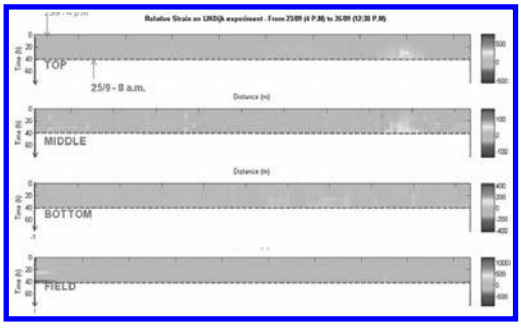

Figure 6. Relative strain measured on Sept. 25th 08:00 a.m. with the TenCate GeoDetect® system, 2.5 days before dike collapse: 0.03% on the top strip, 0.005% on the middle strip (geophy*Consult* data processing).

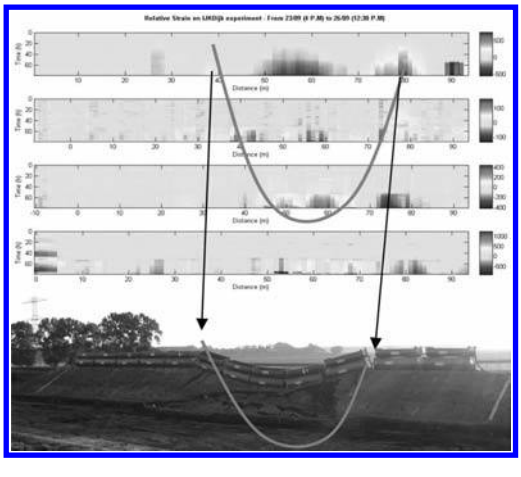

Figure 7. Relative strain measured on Sept. 26th 04:00 p.m. with the TenCate GeoDetect® system, 1 day before dike collapse: the tension zones on each strips draw the boarder of the slope slipping area (geophy*Consult* data processing).

4 PERFORMANCE EVALUATION OF DAM INTERNAL EROSION DETECTION

Another series of 4 experimentations was carried out in 2009 during the second IJkdijk-Piping experimental project for the detection of the early signs of internal erosion at the interface between a clayey dike and sandy erosive subsoil. Full scale levees were built and brought to failure with two explicit goals: to increase the knowledge on levee behavior and to develop and test new sensor technologies for early warning systems under field conditions. Four experiments have been carried out, aiming at failure caused by under-seepage erosion (piping). The clay levee with a height of 3.5 m and slopes of 1:2 was built on top of the sand, 15 m long and 12 m wide. To create erosive channels, water head was increased step by step up to 3 m maximum, to obtain sufficient hydraulic forces to generate an internal piping process. The downstream toe of the dike was inspected visually and continuously to detect water outlets, with or without soil transport. The test stops when the dike collapses (Fig. 8).

As for the macro-stability project, several detection methods were tested. Among them, the GeoDetect® system was installed in the test number 4: three textile sensor strips embedding fiber optics for both temperature and strain measurement were buried 10 cm into the sandy subsoil lengthwise the dike at three different locations ITA0, ITA1 and ITA2 from the downstream toe of the dike as indicated in Figure 9. The temperature optical line was connected to Raman Scattering instrumentation,

Figure 8. Downstream side of the IJkdijk experimental dyke a few minutes before collapse with clear view of the piping chanel and sand erosion at the toe The coloured labels at the toe indicate the observed flows, red with sand transport, blue only water (from Stiching IJkdijk).

Figure 9. Location of the 6 GeoDetect® strips into the sand below the IJkdijk experimental dike.

Figure 10. Installation of the GeoDetect® sensor strips embedding both strain and temperature fiber optics 10 cm into the sand layer before constrution of the experimental dike.

the strain optical line was connected to Stimulated Brillouin Scattering instrumentation. Even though the right detection place would be the interface between the sand subsoil and the clayey dike, as it is where the higher hydraulic gradient occurs, the strips were only allowed to be buried into the sand so as not to disturb the erosion process to be studied during this experimentation.

Three other strips with only temperature optical fibers were installed further up on the upstream toe of the dike at the positions ITA3, ITA4 and ITA5.

Figure 11 shows the maps of the raw temperatures (left column) and of the relative strains (right column) measured by the GeoDetect® system. The temperature profile is disturbed due to the penetration of a colder water flow inside the dike body at distance of about 12 m from the left upstream corner and an inflection of the channel inside the dike body predicting a channel outlet in an area between 8 to 10 m from the left downstream corner. The strain profile on the left is also slightly modified at the same distances with an elongation of the sensors corresponding to about 500 με or 0.05% (dotted white lines). From the observation of these two parameters, this clearly indicates that the start of the piping channel from the upstream side to the downstream side of the dike took place before December 4th, 2009 at 6 p.m., i.e. more than 15 hours before the dike failure occurs. This is confirmed by the evolution of these parameters till December 5th, 2009 at 11 a.m (Fig. 12), 5 hours before the dikes collapses (Fig. 13).

A post-failure analysis carried out by EDF on a another experimental dike (IJjkdijk Piping test n°2) based on signal processing models developed by EDF that is about to be included in real-time data analysis modules has revealed the presence of precursors 5 days before collapse, i.e. only one day after the start of the experimentation (Beck et al., 2010). It has also been observed from the temperature measurements that the speed of the detection increases when the optical cables are located where

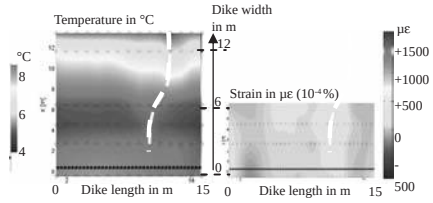

Figure 11. IJkdijk piping dike—Temperature (left) and strain (right) profiles measured on Dec. 4th 06:00 p.m. with the TenCate GeoDetect® system, 22 hours before dike collapse. Heterogeneities appear on the map (white dot line) (geophy*Consult* data processing).

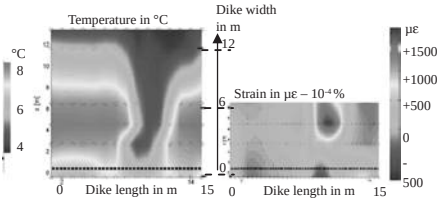

Figure 12. IJkdijk piping dike—Temperature (left) and strain (right) profiles measured on Dec. 5th 11:00 a.m., 5 hours before dike collapse from the raw data on the Ijdijk piping test n°4 showing the extension of the piping chanel bellow the dike (geophy*Consult* data processing).

Figure 13. Collapse of the IJkdijk-Piping experimental dyke after the internal erosion process is completed (from Stiching IJkdijk).

the leaks are foreseen, here at the interface between the sand and the clayey dike, which would be the case for a non experimental work.

This IJkdijk-piping experimentation clearly demonstrates the capabilities of the GeoDetect® system to detect the early stage of a piping process, by analyzing together temperature measurement corresponding to very small leakage rates, less than 1 l/min./m as previously observed in other sites (Artières et al., 2007), and very small soil strain movements less than 0.1%.

5 MONITORING A WET CANAL DIKE

This section describes the monitoring of a 100 m long section of dike along the canal from the Marne River to the Rhine River in France owned by the French waterways (VNF) with the GeoDetect® solution.

This old dike was chosen because leakage was expected in the area, but the localization of the leaks were difficult due to the vegetation on the downstream slope of the dike, and also due to the longitudinal slope of the dike toe collecting large amount of water. The dike is about 3 m high and built with rather homogeneous clayey soil. In this section, it was decided to remove from 20 cm to 40 cm thick of the organic top soil along the downstream slope, before installing four textile sensor strips at different level: top, middle, toe and 2 meters from the toe (Fig. 14). The strips are embedding fiber optic cables both for temperature and strain measurement (Fig. 1). The optical lines were connected as a loop to the Raman and Brillouin distributed scattering instrumentation. After installation of the sensors strips, a filtration geotextile was laid on the entire slope and covered with granular material and organic soil, 20 cm thick at the top and 1 m thick at the toe. The data acquisition took place for about 3 months during the summer 2009.

The mapping of the temperature data is shown on the Figure 15. A temperature anomaly is visible at a 20 m distance on the 3 lowest strips, from the "middle" to the "field", and at about 40 m on the "field" strip (red arrows). For a better data interpretation, a data analysis was carried out using the EDF signal processing model resulting to dissimilarities to daily temperature (Beck et al., 2010). This data treatment confirms one big temperature anomaly on the "bottom" and "field" strips at a distance of 30–40 m (Fig. 16 & 17), which was less obvious on the raw data mapping.

Also visible is the anomaly at 15 m (field) and 57 m (top). It can be observed on Figure 16 that the signal at 40 m on the "field" and "bottom" sensors is not permanent: the cause could be the change of water level into the canal.

The strain mappings are drawn on Figure 18. As "top" sensor is only 20 to 30 cm deep below the surface, the strain data has been corrected with temperature data measured on the other line. The other strips are deeper into the soil and the

Figure 14. Canal form the Marne to the Rhine river. The section with 4 GeoDetect® strips.

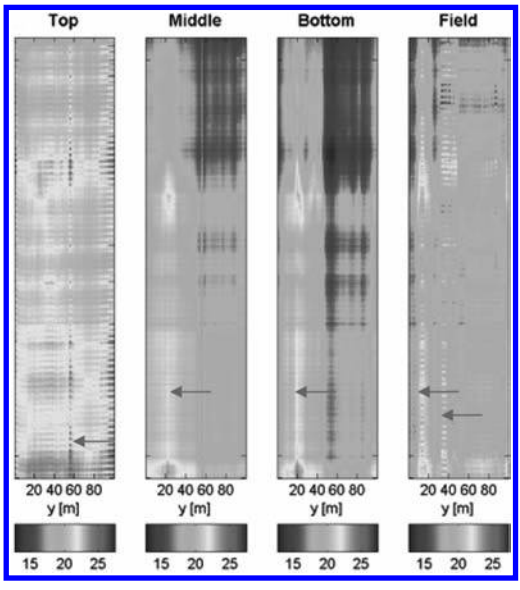

Figure 15. Canal Marne to Rhine rivers. Maping of the temperature raw data (in °C) on the four sensor strips from June 12th, 2009 (on top of the figure) till August 24th, 2009 (bottom of the figure) (geophy*Consult* data processing).

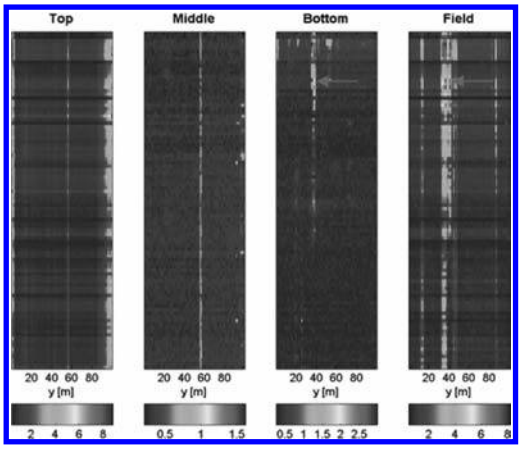

Figure 16. Canal Marne to Rhine rivers. Maping of the dissimilarities from the daily temperature data (in -) on the four sensor strips from April 17th, 2009 (on top of the figure) till August 28th, 2009 (bottom of the figure) (geophy*Consult* data processing).

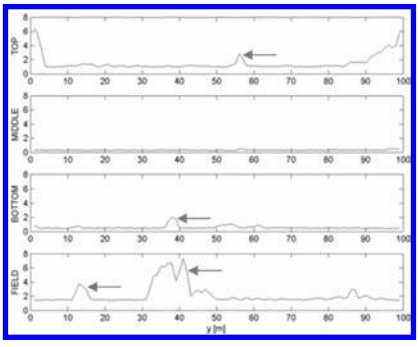

Figure 17. Canal Marne to Rhine rivers. Maping of the dissimilarities from the daily temperature data (in -) on the four sensor strips on April 25th, 2009 (geophy*Consult* data processing).

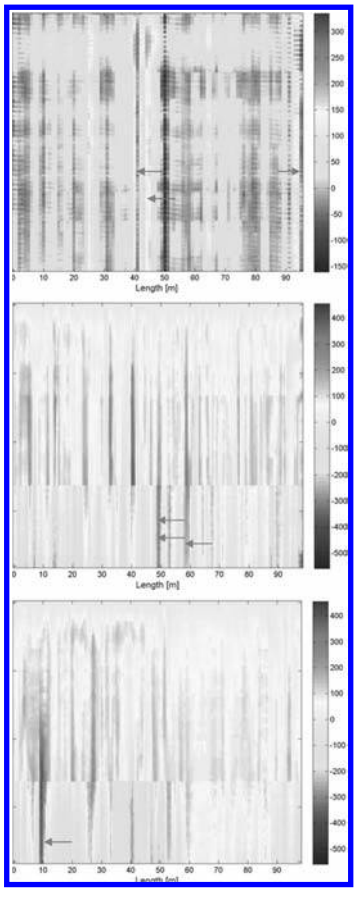

Figure 18. Canal Marne to Rhine rivers. Maping of the strain raw data (in με) on the "Top", "Middle" and "Bottom" sensor strips from June 27th, 2009 (on top of the figure) till September 30th, 2009 (bottom of the figure). (geophy*Consult* data processing).

temperature influence is negligible. The mappings show little movement of the dike which indicates that the observed leakages are not acting on the dike stability. The highest tensile zones, about 0.03%, appears at the "top" line at 40–45 m from June 27th, corresponding to a settlement that can be visually observed at the crest of the dike. Another one is seen at the "bottom" line at a 10 m distance whose magnitude increases with time. Lighter tensile areas are observed on "middle" line at 50–60 m.

There is no direct correlation between these tensile zones and the leakage observed previously. The measured tensile strains may correspond to slight field movements. This analysis shows that there are local differential settlements inside the dike, but no general movement affecting the dike stability.

6 CONCLUSION

Monitoring old and new dams has become a very important issue for their maintenance and the safety. Solutions based on the fiber optic technology demonstrated over the past 5 years have proven their performance in detecting the early signs of dam ageing. Among them, the GeoDetect® monitoring system, with the embedment of several optical fibers for both temperature and strain measurement is very sensitive to detect small temperature changes as little as 0.1°C, small leaks from 0.1 l/min/m and light soil strain as little as 0.02%. Combined with the proper design, installation and a deep raw data analysis from robust signal processing models it is shown to be a powerful early warning solution. It provides dam owners with unique information that drives risk reduction and optimizes the cost of their maintenance strategy.

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Experiences in distributed fiber optical temperature measurements of dams

Experiencias en el control distribuido de temperatura con fibra óptica en presas

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ABSTRACT: Temperature measurement has been used in hydraulic and geotechnical engineering with different purposes for many years. In 1996, distributed fiber optical temperature measurement was used for the first time in hydraulic engineering and developed to an effective tool for dam monitoring. Since then, this technology has been installed in many concrete and embankment dams in order to measure the thermal distribution in the dam body and to detect leakage. In contrast to ordinary thermometers that provide discrete data in selected locations, fiber optical sensors allow temperature measurement every running meter along the cable route with one single measurement. In this article, the functionality of distributed fiber optical temperature measurement is explained and examples of installed systems are shown. Concrete temperature control in a gravity dam and leakage detection of different types of sealing elements in embankment dams are highlighted.

RESUMEN: Desde hace muchos años se utiliza el control de temperatura en la geotécnia y en obras hidráulicas con diferentes objetivos. En el año 1996 se utilizó por primera vez el control distribuido de temperatura con fibra óptica en obras hidráulicas que se convirtió en una herramienta muy efectiva para auscultar presas. Desde entonces, se ha instalado esta tecnología en varias presas de gravedad y de materiales sueltos con el fin de controlar la distribución térmica en el cuerpo de presas y para detectar filtraciones. A diferencia de termómetros convencionales que aportan un control discreto de unos puntos determinados, la fibra óptica permite obtener la temperatura en cada metro a lo largo del tendido del cable con una sola lectura. En este artículo se explica el funcionamiento del método de control distribuido de temperatura con fibra óptica y se presenta varios ejemplos donde fue instalado con éxito. Entre ellos se destacan el control térmico del hormigón en una presa de gravedad y el control de filtraciones de diferentes sistemas de impermeabilización en presas de materiales sueltos.

1 INTRODUCTION

Temperature measurement has a long tradition in dam engineering. For more than 50 years it is used to control the hydration process of concrete during construction. But also during operation, concrete temperature data of dams helps to interpret dam behavior. Especially arch dams are very susceptible to temperature variations. The most common thermometers that have been installed in order to control concrete temperature are thermocouples and thermistors.

Another field of application for temperature measurement in dam engineering is leakage detection. In this case, temperature is used as a tracer for identifying flow paths. This technique is applied in dam foundations, sealing systems and inside the dam body of embankment dams.

In the 1980s, Distributed Fiber Optic Temperature measurement (DFOT) was developed, allowing the measurement of the temperature distribution every running meter along the cable. During the last years, this technique has constantly been improved and offers nowadays a very high accuracy in temperature measurement and spatial resolution.

In the last decade, DFOT has been installed in many dams and other hydraulic structures, offering new possibilities in monitoring by providing distributed data.

In chapter 2 fundamentals of DFOT monitoring systems are explained, and in chapter 3 examples of realized installations and measurement results are shown.

2 DISTRIBUTED FIBRE OPTIC TEMPERATURE MEASUREMENTS (DFOT)

2.1 *Distribute Temperature Sensing (DTS)*

The DTS is based on the temperature-sensitive properties of the fiber which itself represents the sensor. An optical impulse is sent into a fiber integrated in a cable, using a powerful laser. The signal is backscattered with low intensity at every fiber position. Beside the main part of the backscattered light (Rayleigh) there are additional peaks of low intensities (Raman and Brilloiun). The frequency shift and in a certain amount the intensity of the Brillouin Light depends on both temperature and strain at the scattering point. The widely used Raman—systems are using the fact that the intensity of the so called Anti Stokes part in the Raman Light depends on the temperature at the scattering point. Hereby no strain in the fiber is allowed. The distance from the measured point to the laser can be determined by the runtime (time domain OTDR) or by the frequency (frequency domain OFDR) of the light pulse. The cycle time for one distributed temperature measurement ranges from seconds to minutes. One measurement delivers temperature values distributed along the cables with a spacing of 0,25 to 1,0 m. The resulted temperature readings can reach an accuracy of up to ±0.2°C and even better. In dam engineering fiber optical measurement systems, which are using the Raman Effect, are under operation successfully for more than a decade.

2.2 *Leakage detection*

In hydraulic structures, specifically in embankment dams and dikes, the internal temperature field is a function of the flow field. External temperature variations propagate within the dam body by means of conductive and convective heat transport processes, influencing the internal temperature distribution. This close interaction between the flow field and the temperature field allows using temperature as an easily measurable parameter to detect variations in the seepage pattern. Distributed Fibreoptic Temperature (DFOT) measurements allow comprehensive temperature monitoring and consequently comprehensive leakage detection.

The monitoring system is typically implemented by two major approaches: the gradient method (passive method), which uses temperature as a tracer to detect anomalies in the flow pattern, and the heat-up method (active method) describing presence and movement of water in soil by evaluating the thermal response caused by the heat pulse.

2.3 *The heat-up method*

The heat-up method requires an adequate distributed heat input along the cable for a certain time interval. A.C. or D.C. voltage produces the required linear heat input if applied to the copper wires integrated in a heat-up cable. The heat-up method exclusively evaluates the temperature difference between the reference stage before the heating is started and the heated stage. The temperature increase in the cable depends on the thermal properties of its surroundings. In the absence of flow around the cable, the temperature increase is dominated by conduction. Dry and saturated conditions can be differentiated. If the relation between the water content and the thermal conductivity of the surrounding material is known, the Heat-up Method can estimate the moist content. In case of flow around the cable, the conductive heat transport is surpassed by the more effective convective heat transport. The heat input from the cable is therefore transported away more quickly and a steady temperature state is reached. Consequently, the sections with seepage water flow show a much smaller temperature increase than those without.

2.4 *Design and installation*

Like with all monitoring systems the installation of the sensor is a crucial point regarding the functionality of the whole system. Losses caused by damage during construction or due to improper design may severely disturb the measuring results. Therefore it is important to adjust the cable layout to the way of construction and the construction equipment used, so that the cable can be installed easily without interfering the construction process and any damages of the cable or unnecessary fibre splices are avoided. The design of the cable should be chosen that it will meet requirements of the monitoring system in terms of durability, strength, redundancy and heat input. For data

interpretation and presentation it is essential, that the cable location is known at specific points, so that the measured temperature along the cable can be related to the actual location of the cable in the dam structure. Therefore a durable and easy visible length marking should be stamped on the cable coating and the position of the cable should be surveyed in regular intervals and at any turnaround. To avoid damages the cable should not be placed directly in crushed material. It is recommended to use some cushion material around the cable which has similar hydraulic properties to the surrounding material.

3 EXAMPLES OF APPLICATION

3.1 *Concrete temperature control*

Monitoring of concrete temperature with DFOT technology has been applied in Turkey (Birecik), Jordan (Wadi Wala, Wadi Mujib), China (Shimenzhi) and Brazil (Fundao). Besides temperature monitoring of the development of hydration heat regarding the quality control, thermal characteristic values as heat conductivity, heat capacity and thermal diffusivity could be determined for the first time with the heat-up method at Fundao dam. In the following, the installed system at Shimenzhi Dam is described.

The Shimenzhi Arch Dam was completed in 2001 with a final height of 109 m, being the second highest RCC arch dam in the world. It is designed as a very slender structure with about 30 m thickness at the base and 15 m at mid-height, exposed to extreme temperature differentials due to the extreme continental climate (Summer: +30°C; Winter: –20°C).

Because of these considerable temperature changes, tensile stresses will occur with the consequence of high crack building potential. So, the temperature control of the RCC during the construction phase plays an important role in order to avoid the formation of cracks.

During the construction, approximately 300 m of fiber optical cables for the distributed temperature monitoring was installed. The design of the fiber cables layout is shown in Figure 1.

The evaluation of the measurements at Shimenzhi-Arch-Dam show various comprehensible effects, which prove the suitability of DFOT applied in RCC. In Figure 2, the temperature distribution along the cross section of the dam is shown. The continuous line represents the DFOT measurements and the single points temperatures measured with conventional temperature gauges 4 m under the fiberoptic cable. It can be seen that both systems provide temperatures with very similar magnitudes.

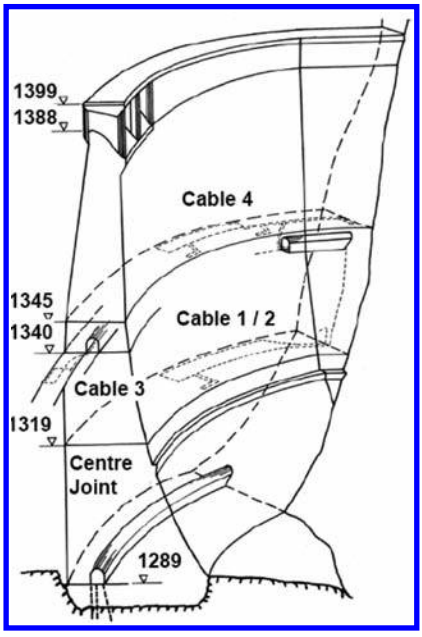

Figure 1. Design of the DFOT monitoring system at Shimenzhi Arch Dam.

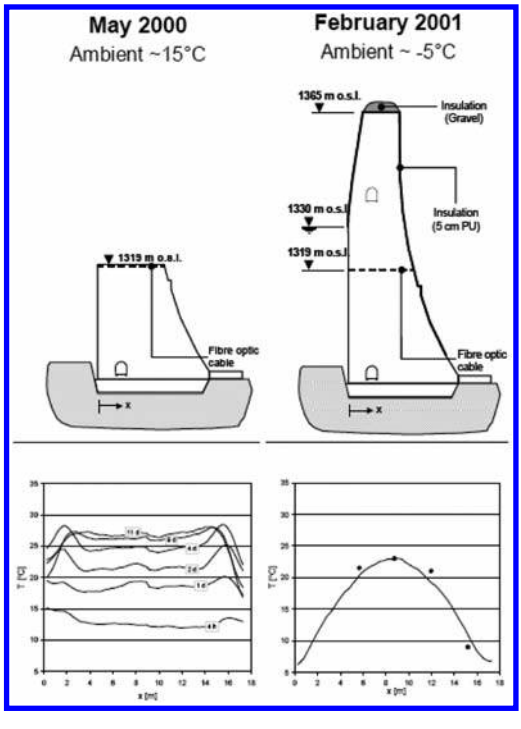

Figure 2. Concrete temperature distribution obtained with DFOT and conventional thermometers during different construction phases.

3.2 *Monitoring of an asphalt surface sealing*

Exemplarily, the layout and results of a system installed to monitor a surface the sealing system of the Ohra dam (Germany) is shown. It is a 60 m high embankment dam with an asphalt surface sealing. Increased leakage indicated deterioration of the existing sealing layer. In particular a 30 m long and in parts several centimeters wide crack was detected at the transition region to the right abutment. In order to guarantee the serviceability of the surface sealing in the long term, a new asphalt sealing layer has been placed on top of the existing one. For advanced monitoring of the critical area close to the right abutment a complementary leakage detection system based on DFOT measurements has been installed. A 30 m wide and 110 m long section is monitored by an approximately 720 m long fibre optic cable which is placed in 21 loops with a distance of 5.60 m between the loops (Figure 3). Generally, the leakage detection system can monitor the watertightness of the whole sealing element. However in this case it was sufficient to concentrate on the area where cracks may occur most likely.

In order to check the functionality of the system, artificial leakage of different intensity has been applied. For that reason a tank wagon has been positioned at the dam crest and connected to the intended infiltration openings. The inflow has been varied between 0.01 l/s and 0.05 l/s. On the basis of the measuring results the progression of the infiltration can be followed easily (Figure 4). Also significant is the dependence of the resulting abnormality of increase of temperature from the intensity of the leakage flow. Additionally it has been shown that even leaks with small inflow (0.02 l/s) can be detected.

3.3 *Monitoring of a concrete facing joint*

A further field of application is the monitoring of joints. In CFRDs the perimeter joint is the critical point in terms of infiltration, because it is where the main leaks happen. Its movement is three dimensional caused by the fact that the plinth is anchored to the foundation rock and the concrete slabs of the face rest on the compressible rockfill. The installation of fibre optic heat-up cables to monitor this joint is a highly efficient application of the leakage detection system. The cable is installed on the mortar bed below the bottom copper water stop (Figure 5).

A DFOT leakage detection system to monitor the plinth is installed at the Merowe CFRD in Sudan. Similar systems are also installed at the Siah Bishe CFRDs in Iran, the Kalivac CFRD in Albania and the Nam Ngung 2 CFRD in Laos.

The gain in information of the leakage detection system for monitoring the perimeter joint can be

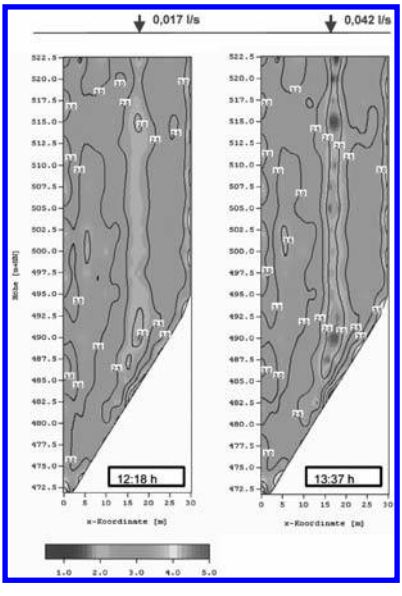

Figure 4. Measuring results after leakage simulation at Ohra dam.

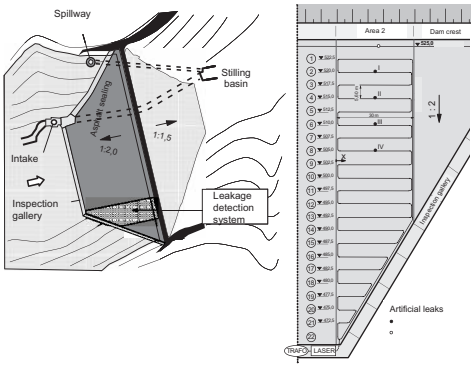

Figure 3. Design of the DFOT monitoring system at Ohra dam.

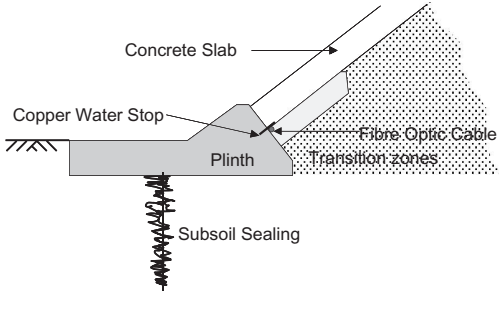

Figure 5. Location of the fiber optic cable at the perimetric joint.

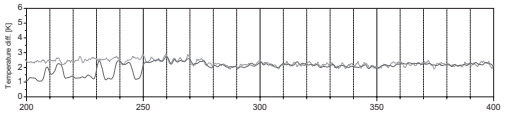

Figure 6. Comparison of temperature increase along a section of the dam (green line: zero measurement/red line: measurement during the appearance of leaks).

shown best on the basis of the monitoring data of a dam in the Southern Hemisphere. The 30 m high and 2450 m long dam with an asphaltic concrete facing is founded on clayey soil with sand lenses and boulders. Sealing of the foundation is achieved by a cut-off wall. A total of 2,850 m of fibre optic cable were installed to monitor the connection of the cut-off wall with the asphaltic concrete facing. After impounding the reservoir, seepage water was pouring out of several drainage outlets. The results of the performed fibre optical temperature measurements to detect the leaks are shown in Figure 6. The temperature differences have been calculated by subtracting the temperature along the cable in the unheated state from the temperatures along the cable measured while heating the cable. The results of the zero measurement with an empty reservoir are displayed as a green line. The results of the measurement after seepage has occurred are shown by the red line. The temperature increase is not uniform along the length of the cable. Locations of significantly lower temperature between position 200 and 250 indicate several leaks in this section. Outside this area leaks can be excluded for the displayed section.

3.4 *Monitoring of geotextile membranes*

Installing an impervious synthetic geomembrane over the upstream face to stop leakage has been applied at ceveral CFRDs. The geomembrane is installed in sheets which are watertightly joint by heat welding. It is anchored at all peripheries generally with mechanchical anchorage, designed to withstand the hydraulic head applied. Fibre optic leakage detection systems to monitor the geomembrane have been applied at several dams. Among others are the Winscar Dam (UK), the upper reservoir of the pumping storage scheme Waldeck I (Germany) and the Kadamparai Dam (India). All installed systems excel as reliable and durable monitoring systems.

4 CONCLUSION

Distributed Fiber Optical Temperature measurements (DFOT) contain a number of important technological advantages such as high information density, the suitability for rough site conditions and the simple and flexible installation of the cables.

This measurement system proofed its suitability even under the harsh site conditions of RCC dam sites, where it has to be installed during the rapid construction process being exposed to high vehicle and compaction loads.

It is also a very efficient tool for leakage detection in different kinds of sealing systems in embankment dams and may allow detecting of internal erosion processes at an early stage. It is also possible to measure filter velocity in porous media.

Current research projects aim on the application of distributed fiber optic measurements in deformation measurement and crack detection.

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Rehabilitation and improvement to the monitoring equipment and data processing for the 272 m high Enguri arch dam in Georgia (CIS)

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ABSTRACT: Enguri dam in Georgia is the highest arch dam in the world at 272 m high. The first full impounding of the reservoir was performed in 1998. Due to some years of civil unrest following the break up of the former USSR in 1990, a lack of maintenance resulted in the Dam and Power House falling into a state of neglect thus poor reliability and safety of the scheme was evident. In addition to this deficient maintenance, all the monitoring equipment was in poor state: all electrical instruments and wiring had been stolen, many mechanical instruments were broken or badly rusted and geodetic instruments were missing to perform measurement campaigns.

In 1995 the Georgian Government required the financial assistance of the European Bank for Reconstruction and Development (EBRD) to assist in the rehabilitation of the scheme. Initially a grant from the Swiss State Secretariat for Economic Affairs (SECO) allowed the installation of a new dam monitoring equipment. In a second phase starting in 1999, the EBRD financed a more global rehabilitation project including reparation of the pressure tunnel, improvement of the grout curtain and the hydro mechanical equipment in the dam and the power plant, but also a completion of remaining monitoring works not done in the first phase.

The goal of these rehabilitation works focused mainly in performing many drilling works for the installation of new piezometers and extensometers but has consisted also in putting in place a data acquisition and processing system permitting a rapid interpretation of the dam behavior and a remote control access. The selected software was CESmon, which has been used already by several operators in Switzerland. This article focuses on the rehabilitations of the complete instrumentation and installation of the automated remote data acquisition and processing system.

1 INTRODUCTION

1.1 *General description of the scheme*

Enguri river basin is located in western part of Georgia, from the southern slopes of the Main Caucasus Range to the Black Sea. The Enguri River 221 km long originates at the Ukhvan (5058 masl) and Shkhara (5201 masl) glaciers on the southern slope of the Main Caucasus Range. The Enguri is a river with a high water flow of a compound-feed type, primarily glacier and snow-fed. The rated maximum discharge at 0.01% probability is 2500 m^3/s. The natural average discharge at dam site is about 150 m^3/s.

The estimated hydropower potential of the Enguri river is $21*10^9$ kWh per year, of which the economically feasible potential is $10–11*10^9$ kWh per year. First plans to use the Enguri rivers hydropower potential resources were prepared in the 1930s. Later these plans were modified and in the 1950s it was decided to construct a chain of dams and power plants including itself the Enguri HPP (Dam height 271,5 m, Power House capacity 1300 MW) and Vardnili HPP (4 HPPs with total capacity 340 MW). Construction of these power plants was executed between 1960 and 1986. The present chain of power plants is generating about half of mentioned potential resources ($5.5*10^9$ kWh per year).

The Enguri HPP was designed to generate electricity to deliver peak electrical energy to the electrical network of the former USSR. The scheme consists of two main parts: the dam and its appurtenant structures, which are connected by a 15 km long 9.5 m diameter pressure tunnel and the power plant. Two more dams were planned upstream of Enguri and construction of one of them, Khudoni, was started but the project abandoned after substantial excavations of the arch dam foundations, river diversion scheme and the underground power house.

The Enguri double curvature arch dam is the highest dam of its type in the world. At present,

this power plant provides one quarter of Georgia's total generating capacity and about 45% of the total system energy.

Enguri dam body was completed only in 1984, several years after the power production at reduced head started (the units were commissioned in 1978–80). Treatment of the dam foundations and other works continued until 1988, the first year in which the reservoir was allowed to fill to its maximum level. In 1990s for economic reasons and particularly due to the civil unrest in the region, after the break up of the Soviet Union, the structures have suffered from neglect and the lack of maintenance. Among the major parts of the scheme affected was the dam hydromechanical equipment, the power plant, but also severe damage and vandalism occurred to the all the monitoring system.

1.2 *Recommendations for monitoring rehabilitation*

It was, therefore, in 1995 that Georgian Government and the scheme's Owners Engurhesi Ltd (the former state entity SAKENERGOGENERATSIA) requested the European Union and the European Bank for Reconstruction and Development (EBRD) to provide financial support for dam rehabilitation.

After a mission of experts of the E7 group in April 1995, they came to the conclusion that the dam safety, the power plant and their facilities were causes of great concern. In particular it was noted that the state of the Enguri Dam facilities did not meet minimum safety requirements, that vital components related to safety were not operational and that a dangerous situation beyond the utility's control was prevailing.

A further analysis of the situation was performed by the JV STUCKY-Electrowatt in August 1995 and showed the most urgent activities and the necessary equipment to improve this crucial state of the dam. This report was handed over to the Swiss Federal Office for Foreign Economic Affairs (FOFEA now called SECO) and the European Bank for Reconstruction and Development (EBRD) in September 1995. The report proposed the immediate reactivation of the dam monitoring, to assess the actual state and behavior of the dam, as until this was known no further rehabilitation and safety works could be designed as badly needed.

Figure 1. Enguri double curvature arch dam—the highest dam of this type in the world at 271.5 m.

This first monitoring equipment rehabilitation and reinstallation took place from 1996 to 2001. The main objectives of this first phase works funded by SECO can be summarized as follows:

- Rehabilitation of the dam monitoring system and also the improvement of the overall surveillance of the structure including visual inspections of the dam and with an evaluation of instrument records.
- A transfer of technology and know-how by training local monitoring staff to process and screen the data, in order to evaluate the recorded results and assess the current safety of the dam and also the future assessment.
- An assessment of the dam behaviour based on the old and new monitoring results.

Following this first rehabilitation project, a second phase of rehabilitation started in 1999. The EBRD financed a more global rehabilitation project of the whole dam, power tunnel and the power house. The following works were carried out on the dam site from 2005 to 2008: grouting works to extend the grout curtain; concrete and shotcrete works to line galleries and to secure access to the dam foundation, portals and intake; full renewal of the gravity and pumping drainage systems of the dam foundation; further upgrading of the dam monitoring system; slope stability measures close to the power intake; installation of a new stoplog for the low-level outlets; repairs to headworks valves; valve chamber waterproofing; etc. The main goals of the works were to reduce seepage, uplift and internal erosion of the abutments, improve the drainage system and secure the access and the operation conditions.

The goal of the monitoring rehabilitation works focused mainly in performing many drilling works for the installation of new piezometers, additional extensometers and plumblines. It has consisted also in putting in place a data acquisition and processing system permitting a rapid interpretation of the dam behavior and a remote control access. The selected software was CESmon, which has been used already by several operators in Switzerland and elsewhere in the world.

2 FIRST DAM MONITORING REHABILITATION FROM 1996 TO 2001

2.1 *Initial state*

The missions carried out in November 1995 revealed the poor state of the dam and its monitoring equipment. The Table 1 below resumes the monitoring state as it was discovered:

In more details, some of the most critical points observed during the first visits of 1995 were:

- First of all the safety of people working for monitoring purposes in the dam was not ensured at all. For example, in every block there are vertical drainage shafts on the invert of the inspection galleries which were left open. The risk was really high that staff could fall in them, especially that galleries were not illuminated at all. Several other dangers were existing such as broken metallic stairs, missing parts of balconies, steel bars coming out of the walls, etc. … Generally it was evident that the dam was never completed with some galleries even being blocked by old forgotten drilling rigs and formwork.
- The plumbline equipment was in really bad state, all the reading stations were highly rusted, the wires were bent or even broken, and in addition the reading instruments had been stolen and a temporary one was used from another dam located quite far from Enguri. As for many other metallic instruments or equipment, the steel used was probably very poor or badly protected and all these metallic parts were found very corroded even after about 10 years of being used on the dam.
- The knowledge of uplift and pore pressure of the foundation was not clear as most of the manometers which equipped the more than 200 piezometers were stolen or broken.
- A huge quantity of water was flowing out of the drainage galleries (a total of more than 1 m^3/s), but the real amount was not well known as no gauge stations at all was built to perform precise measurements. The readings were performed by measuring the speed of the water flowing on the galleries invert.
- The electrical monitoring equipment, which was one of the most ambitious and extensive ever see in a dam with more than 6000 instruments installed was completely out of order. All the cables had been cut at the surface of the galleries walls and the connection boxes had disappeared. It was therefore impossible to determine the concrete temperature of the dam for instance.
- The state of the geodetic network and survey instruments was also poor and consequently the few existing measurements were totally inaccurate.

Table 1. List of the monitoring equipment state in 1995.

Monitoring equipment	State in 1995
Plumblines	Deficient and badly damaged; reading instrument missing
Piezometers	Manometers stolen or completely out of order and never calibrated
Meteorological instruments	Missing since 1992
Geodetic network	Inappropriate, no redundancy
Geodetic instruments	Missing since 1992
Leveling system	Partly destroyed, instruments missing
Drainage gauge stations	Were never completed and measured
Reservoir level measurement	Measured manually once per day
Strong motion recorders	Probably stolen could not be found
Electrical instruments for measuring of temperatures, joints and stresses	Electrical cables cut, connecting boxes destroyed
Illumination in the galleries	No lights installed completely dark underground

Basically the owners of the worlds highest arch dam had lost complete control of the structures behavior and thus the safety of the dam.

2.2 *First stage of the dam monitoring system rehabilitation project*

Between Swiss and Georgian governments an overall grant of 3 Mio CHF was agreed in 1996, covering the involvement of Swiss suppliers, contractors and the consulting Engineers. The Georgian contribution was agreed as to cover the wages of local staff and contractors. The works to be performed locally were cleaning of galleries, installation of monitoring equipment, erection of the survey pillars, drilling work and civil work for the plumbline system and building the new drainage gauge stations.

Concerning the Swiss contribution, more than one third of the grant was used for the drilling works and delivery of material and machines for drilling. Another third part covered the costs for delivery of all the monitoring equipment, geodetic material but also some office material and computers to help the local staff to work more efficiently and improve the transfer of know-how.

The main works performed are summarized in the Table 2.

A perfect example of the improvement brought to the monitoring system can be shown by exposing the case of the plumblines. Below in Figure 2 please see a chosen dam elevation with the complete new network of plumblines. It shows all the existing replaced plumblines and the location of the 8 invert plumblines drilled in the foundation. The Figure 3 shows the state of the existing equipment and the newly built reading stations.

The replacement of all the plumblines and the construction of new perfectly stable concrete reading stations have significantly increased the precision of measurements. The Figure 4 shows the upstream/downstream movements measured at the plumbline 26 at elevation 474 masl. In gray are the movements measured before the replacement of the plumbline equipment. It is clear that the original scattering of points has really improved, and that the new records are all located on a much narrower thus believable envelope of points.

As the dam was strongly reinforced in various places as part of the Soviet way to combate the seismic loads, the drilling of the new plumbline shafts became much more complicate than what was foreseen and big delays have occurred in this work which have prevented the complete installation of all inverted plumblines till the end of the project.

Similar difficulties have occurred in the drilling of extensometers holes. In addition the upstream drillings were strongly hindered by the high water pressures and the difficulties for the Georgian local team to cope with this problem. To be noted that the installation and drilling when the reservoir has been impounded for such a high dam is very difficult.

The complete reconstruction of the geodetic network and the use of very modern equipments such as Leica total Station have permitted to get results of the dam movements perfectly comparable with the measurements performed with the plumblines. Even if the survey works are often rendered difficult by the strong winds and long distances of measurements of such a big dam, the quality of the results are now perfectly comparable with any modern dam.

Table 2. List of the monitoring equipment installed till 2001.

Monitoring equipment	Worked performed till 2001
Existing plumblines	Full replacement of all parts except metallic shafts. Construction of concrete walls as new reading stations
New plumblines	Drilling of 8 holes in the pulvino and foundation to install invert plumblines (4 had still to be installed after 2001)
Piezometers	Delivery of 210 new manometers & taps
Meteorological instruments	Complete new automatic but mechanical weather station
Geodetic network	Complete new network and construction of stable pillars
Geodetic instruments	Delivery of a Leica Total station and various other survey components
Leveling system	Rehabilitation and delivery of missing parts
Drainage gauge stations	Construction of 20 gauge stations with weir or bucket measurements
Reservoir level measurement	Delivery and installation of Rittmeyer high precision water balance with remote readings of lake level
Strong motion recorders	Delivery of 11 accelerographs with 18 bit-sensor and recorder fully interconnected
Additional equipments:	
Extensometers in the foundation	A third part of the delivered multi-rod extensometers are installed
Vertical clinometers	To replace the missing plumblines of the upper parts of blocks 7 and 32, clinometer stations are installed to record the dam movements
Horizontal clinometers	For a better follow-up of the dam foundation & pulvino, deformations are checked along the thickness of the sections
Turbidimeter	Portable instrument delivered for seepage measurements
Electrical instruments for measuring of temperatures, joints and stresses	Recovery by a local institute of the concrete temperature measurements
Illumination in the galleries	Lightning was installed at the reading stations

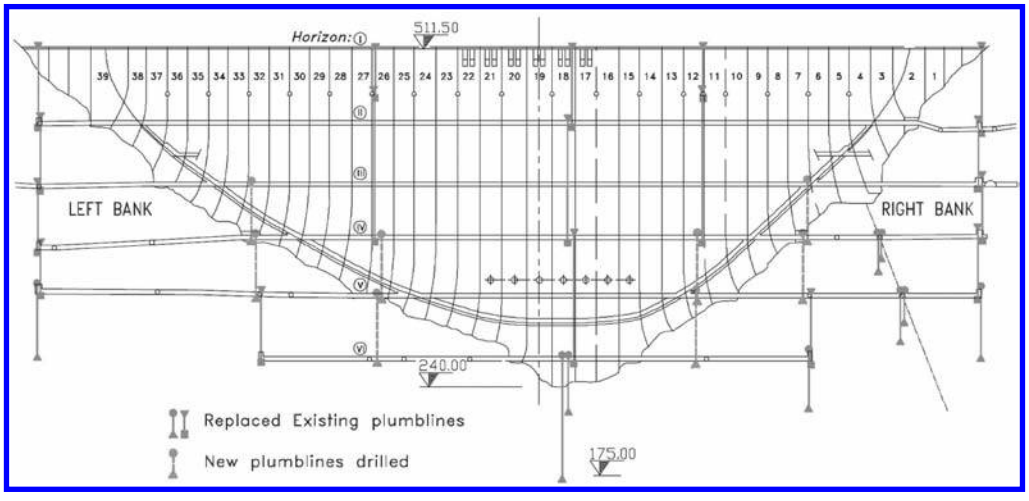

Figure 2. Upstream elevation with the location of all plumblines.

Figure 3. A plumbline reading station state before and after rehabilitation.

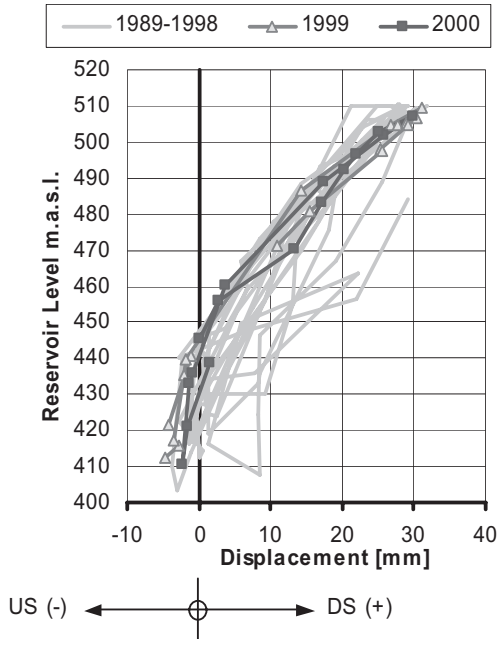

Figure 4. Upstream/downstream movements measured at the plumbline 26 at elevation 474 masl.

The objective of this first rehabilitation project was also the training of the local staff to perform geodetic survey and reading of the dam monitoring measurement as well as to maintain in good state the equipment and perform correctly the dam behaviour analysis.

During the training it was ascertained that the staff could follow the course and recognized the correlations of the systems and the evaluation procedure. The main insufficiencies were found not to be technical but rather economical and possibly motivational. This manifested itself as a tendency towards reluctance to take initiative, almost certainly a consequence of working in the strict hierarchical system of earlier soviet times, and was accentuated by uncertainties about when and whether salaries and wages would be paid.

Unfortunately, it was discovered a few years after the completion of the project that most of the trained staff had left the institute and clearly the transmission of an important part of the know-how was not performed or lost. Only the survey team, which became independent, has continued to

do their job and performed regularly and precisely the work. Nevertheless, other monitoring tasks have continued in more or less good conditions till the second phase of the monitoring rehabilitation project as described below.

3 SECOND PHASE OF THE DAM MONITORING REHABILITATION FROM 2005 TO 2008

3.1 *Completion of the first phase and new instruments*

In the frame of the extensive complete dam and power plant rehabilitation works, the completion of the remaining drilling works not performed during the first phase and the installation of some additional monitoring instrument have been done.

Concerning the outstanding works to be done from first phase, two remaining drillholes for invert plumblines and 2/3 of the foundation extensometers have been installed. These works were performed by Georgian staff of the main contractor SAKHYDRO under the technical assistance provided by Boartlongyear under the supervision of STUCKY.

Due to important grouting works for consolidation but also rehabilitation and extension of the grout curtain, some existing piezometers were damaged, other had to be cleaned, but also additional piezometers had to be drilled. Furthermore, to avoid risks of important leakage inside of the foundation galleries all bad state piezometers and moreover most of the piezometers located upstream of the grout curtain were plugged by grouting them as they were not needed. This task has permitted to reduce also significantly the water flows in the galleries by grouting some old broken piezometers bringing high leakage and not giving any useful information.

A full back analysis of the structure could now be done to analyse the structure behavious of the dam under normal and abnormal load cases. For this STUCKY made extensive calibration of the finite element model and compared it to the newly acquired data from the monitoring system that could now be trusted once again.

During this behavior analysis performed during this stage, it was pointed out some additional instruments needed to be installed or replaced, these included.

3.2 *Setting up a data acquisition system*

During the performance of the complete dam behaviour analysis by finite elements, with all the "new" data acquired from the monitoring instruments installed since 1998, it was clearly observed many inaccuracies in the databases prior to this.

When performing this behavior analysis a lot of time was lost and most of the inaccuracies in data collection could have been avoided. The most common reasons of errors in the databases were the following:

- Errors in performing or writing the readings on site without any method for controlling the values
- Mistakes when retyping the data in the Excel sheets
- Calculations errors in Excel due to modifications of the tables without enough care
- As several people of the monitoring team are working in the treatment of the measurements, several versions of each Excel sheets exist at different stages of corrections or modifications. This brings to confusion in using the right values for working

Table 3. List of the monitoring equipment installed from 2005 to 2008.

Monitoring equipment	Worked performed till 2008
Concrete thermometers	The results of the old electrical instruments were not adequate. Consequently, thermometers were installed in new drillholes
Reservoir level	The Rittmeyer water balance could measure the lake level between 415 masl and 511 masl. For lower levels, a new less precise pressure cell was installed
Drainage gauge stations	Additional gauge stations were built in order to define more precisely the water seeping in the different galleries of the dam
Meteorological instruments	The low maintenance of the initially installed weather station made it out of order. A new fully automatic station was delivered and installed

– Rapid data analysis was not always done shortly after performing the measurements. This means that the readings are left on a sheet of paper during several months before being typed in a database for evaluation (if not lost meanwhile).

The only solution to avoid all these above problems was to put in place a system permitting a digital data acquisition system with immediate plausibility check of each single value and one single place for the backup of all the data from the dam.

The management company Engurhesi Ltd. agreed to the proposal of STUCKY to acquire and put in place the system called CESmon (Civil Engineering Structure monitoring).

Mainly the delivered equipment was the following:

– A main Windows server with double backup system of all the data and using an Oracle database
– Two efficient workstations connected in a network for the data treatment and upload of data to the server
– Overvoltage equipment to protect efficiently all computer components
– Two portable data reading terminals (PSION terminals)
– All the material to install precise labeling at every reading stations (metallic supports, stickers, …)

The server has been fully configured and tested in Switzerland before being commissioned on the dam site with all the rest of the equipment in spring 2007. From here on the preparation of the configuration of the 500 reading points to be introduced in CESmon started. This work was performed jointly between the monitoring team of Engurhesi and STUCKY.

For each of the 500 reading stations the following work had to be done:

– Recover, check meticulously and correct (if needed) the more than 10 years of data
– Based on a statistical method, define the plausibility limits out of which a reading will not be accepted before additional control. This work needed to establish formulas using external parameters such as reservoir level or temperatures (air and concrete) in order to define the minimum and maximum limits
– A complete renaming of all the stations was also needed to make clearer the work of interpretation. Based on this, the printing and the installation of 500 small supports with bar code stickers, which identify the reading station for the PSION, was done.

From the end of 2007 the system entered in a testing phase to check if all the configuration of all the 500 reading points is working properly. Various small corrections had to be brought to the system until it has started to work at 100%.

The Figure 5 below shows how the PSION makes the identification of each reading station via the bar codes.

The testing phase was also required to train all the staff of the monitoring group on how to use the PSION for performing the readings inside of the dam and type the data on the terminal, on how to download the values on the main server and check the dam behavior, but also on how to react in front of different problems which can occur with electronics (battery charging, resets, reinstall system, etc. …). Various missions were carried out in order to perform a full transfer of know-how. During each mission not only work on the configuration of CESmon was performed but also courses on its management, training on dam behaviour analysis and various recommendations on the management and maintenance of all the monitoring equipment.

One of the great interests of CESmon is that it can be easily configured for various types of users and administrators. On every part of the software different levels of access rights can be set to avoid any risk of changes by unauthorized persons. Once this entire configuration is done, the server can be easily reached from any workstation connected to internet over the world in order to perform data analysis, rapidly evaluate the dam behavior (for an external expert) or simply check the actual reservoir level.

Figure 5. Performing measurements on bar codes at each reading station.

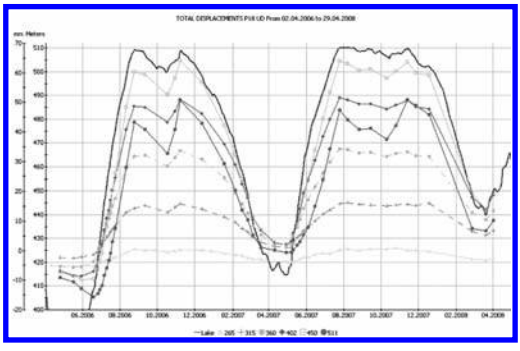

Figure 6. Screen of the total displacements of block 18 plumbline.

CESmon is very well oriented towards the dam Owners, as various fields permit to keep the information on the maintenance of all the monitoring equipment, record the good or bad functioning of some instruments, take easily notes on observations made during a measurement campaign or record precisely who has performed what task during the day for the monitoring works.

Below is one example of a CESmon screen permitting the evaluation of the block 18 plumbline total displacements.

4 CONCLUSIONS

The rehabilitation of a complete monitoring system is not just a replacement of existing old instruments by new ones. Especially in a country were electricity power cuts are common, spare parts are not always easy to get and where theft of material happens often, all the concept has to be thought out in order to deliver the best long lasting material and very resistant portable reading instruments. Except if they can be well secured in niches or solid boxes and well protected against over voltage, the fix electrical reading stations have to be avoided as much as possible. Thus it needs an "action plan" that is made by an experienced dam engineer and who knows the output required from a monitoring system to allow redundant, outdated methods, or instruments to be eliminated and modern ones installed.

The concept put in place in Enguri was as much as possible performed with manual or portable instruments but permitting easy readings and a rapid interpretation of data for a good day by day assessment of the dam behavior. With the training given to the monitoring team and the equipment permitting a simple control of every single reading, any single problem appearing in the dam should be rapidly discovered and re-reading could be easily made if needed. Thus a user friendly and self checking reading system needs to be installed to eliminate costly and misleading errors in the field.

The collaboration and interest in the subject shown during several years working with the engineers of the Engurhesi monitoring group should ensure their interest in continuing their tasks and the good follow up of the behaviour of the Dam. It is important that Georgian Government and the dam owner Engurhesi Ltd continue to give a great importance to the monitoring supervision of the dam by not putting any limitation in financing and maintenance of the of the existing equipment. Thus the importance of a well maintained network of equipment with a motivated and trained team is essential to the dam safety.

The centralization of all monitoring data in one single place in CESmon software has brought much clarity to all the data measured since the beginning of the dam construction. It has obliged the team to make review and elimination of all the unnecessary data tables and reorganize the way of working in one definite standard understood by everyone. Thus an easily accessible and centralized data base is essential to allow the dam supervising engineer access and confidence in the results of the dam behavior that he shall analyze.

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Nondestructive Evaluation of concrete dams

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ABSTRACT: There are a number of Nondestructive Evaluation (NDE) methods applicable to investigations into the condition of concrete dam structures. Included in this paper is a discussion of the methods themselves, the applicability of the methods to different types of dams and structures related to dams, as well as several case histories of the methods in application. The NDE methods to be discussed include Spectral Analysis of Surface Waves, Slab Impulse Response, Ground Penetrating Radar, Cross-Dam Tomography and Impact Echo for imaging conditions inside concrete dams. The methods are applicable to a variety of concrete elements found on many types of dams. These elements include spillway concrete evaluation, spillway subgrade evaluation, dam wall concrete evaluation, and thin-arch dam interior evaluation. Several of the techniques discussed can be applied from the downstream face of a thin-arch concrete dam to evaluate the condition of the concrete through the entire cross-section with no dewatering required. In fact, the Cross Dam Tomography method works best when the dam is as full as possible. Conditions evaluated include freeze-thaw damage, cracking, general concrete quality, and void. The results of the nondestructive evaluation aid in planning repairs and evaluating dam safety.

1 INTRODUCTION

As the world's infrastructure ages, it becomes increasingly important to be able to determine the condition of existing structures, and track changes in condition over time. This is especially true for critical infrastructure elements such as large concrete dams. The long expected life and large mass of concrete dams and spillways tends to make them susceptible to gradual degradation mechanisms which begin in very small ways which are easy to miss, but can accelerate over time. Some of these mechanisms include freeze-thaw damage on the downstream face and crown, seepage under and around outflow pipes and spillways, slow-developing cracks in the dam interior, and erosion due to water flow and weathering. Each of these types of degradation can start out as a very minor problem, which sometimes can be present for years before being noticed. The presence of the initial degradation, however, tends to accelerate future problems. Freeze-thaw damaged surfaces hold more moisture than usual, leading to greater damage. Small cracks in the dam face become stress concentrators which lead to deeper cracking, while cracks which seep can lead to erosion and cracking due to the action of the water. Seepage under a spillway or around an outflow pipe slowly erodes the subgrade, leading to faster flow rates and even greater erosion rates.

The best defense a dam owner has against these threats is periodic inspection. Visual inspections, however, often only find problems after they have developed into major degradation. Some types of damage, such as voids under a spillway slab or cracking in the dam interior or on the upstream, submerged face, are very difficult or impossible to find with only visual means. This is where the use of Nondestructive Evaluation (NDE) techniques enters the picture. The various nondestructive methods available use sound waves, radio waves, and other types of low-level energy to penetrate through the concrete beyond what the eye can see. These methods can locate a void under a spillway, measure the depth and severity of freeze-thaw or other surface damage, and create an image slice through the interior of a dam to show cracks, debonded joints, and other degraded zones. Most of these methods can be done without dewatering the dam, and all are completely nondestructive and nonintrusive. In this paper, several of the NDE methods most commonly applied to dams will be discussed, along with sample results from investigations of a number of dams of various types.

2 NDE METHODS

While a number of NDE methods can be applied to the investigation of the condition of various concrete elements in a dam structure, there are five techniques that are particularly useful in most concrete dam investigations. The first technique discussed is the Spectral Analysis of Surface Waves

(SASW) method, which is used for measuring the condition of concrete versus depth from the test surface as well as for crack depth measurement (Olson et al., 1994). The second method discussed is the application of Ground Penetrating Radar (GPR) to spillway investigations. This method is used to map out voids and water seepage paths under a spillway, and will also provide information as to the spacing and depth of any reinforcing in the concrete for cases where this information is not known. The third method is Slab Impulse Response (SIR) which is sensitive to thin voids below spillway slabs and conduits. The fourth method discussed is Acoustic Tomography (AT) which involves the use of sound wave transmission through a thin-arch dam interior to create a tomographic velocity image slice through the dam from upstream to downstream faces. The AT method is a very powerful and relatively quick way to see beyond just the downstream face to the interior and upstream faces without dewatering.

3 SASW TEST METHOD AND RESULTS

The SASW method is based upon measuring surface waves propagating in layered elastic media and is illustrated in Figure 1. The ratio of surface wave velocity to shear wave velocity varies with Poisson's ratio. However, reasonable estimates of Poisson's ratio and mass density for concrete and other materials can normally be made with only a small effect on the accuracy of the shear wave velocity profile. Knowledge of the shear wave velocity combined with reasonable estimates of mass density of the material layers allows calculation of shear moduli for low-strain amplitudes. Surface wave (also termed Rayleigh; R-wave) velocity varies with frequency in a layered system with differing velocities. This variation in velocity with frequency is termed dispersion. A plot of surface wave velocity versus wavelength is called a dispersion curve. The SASW tests and analyses are generally performed in three phases: (1) collection of data in situ; (2) construction of an experimental dispersion curve from the field data; and (3) forward modeling of the theoretical dispersion curve, if desired, to match theoretical and experimental curves so that a shear wave velocity versus depth profile can be constructed.

Wavelength (λ), frequency (f), and Rayleigh (surface) wave velocity (Vr), are related as follows:

$$Vr = f * \lambda \quad (1)$$

Surface wave dispersion can be expressed in terms of a plot of surface wave velocity versus wavelength. This plot is called a dispersion curve plot. For sound concrete, the dispersion curve plot is flat (constant velocity) versus wavelength throughout the concrete thickness. If degradation or cracking present at the test location, the dispersion curve will have zones of lower velocity. The depth of the cracking or degradation can be estimated from the wavelength range of the dispersion curve.

3.1 *Collection of SASW field data*

Typical SASW field tests are conducted with a pair of matched vibration transducers and an impact source. The transducers are typically displacement transducers or accelerometers mounted to or held in contact with the concrete surface. One example of an SASW data collection system is an Olson Instruments SASW-S test system, consisting of two displacement transducers mounted on a bar at a fixed spacing. The bar is pressed against a given test surface, and impacts made with a small hammer in line with the receivers. A photograph of SASW data collection on a thin arch dam face using rope access is shown in Figure 2.

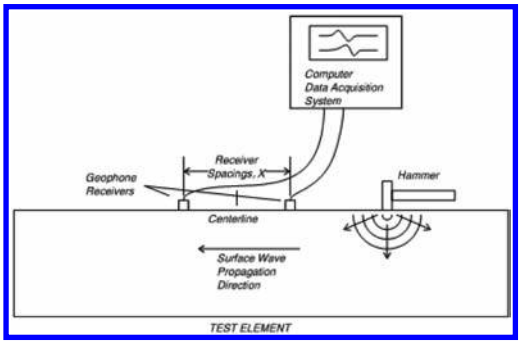


Figure 1. SASW field layout of source, receivers, and collection system for typical SASW testing.

Figure 2. SASW testing on dam face for freeze-thaw cracking depth.

A computerized signal conditioning and data acquisition system is then used to collect the waveforms from the two receivers and record the signals for spectral (frequency) analyses. The phase information of the transfer function (cross power spectrum) between the two receivers for each frequency is the key spectral measurement. All field data are recorded on the computer hard drive for later analysis.

3.2 *SASW experimental dispersion curve processing*

The experimental dispersion curve is developed from the phase data from a given site by knowing the phase (N) at a given frequency (f) and then calculating the travel time (t) between receivers of that frequency/wavelength by:

$$t = N/360 * f \quad (2)$$

Surface wave velocity (Vr) is obtained by dividing the receiver spacing (X) by the travel time at a frequency:

$$Vr = X/t \quad (3)$$

The wavelength (λ) is related to the velocity and frequency as shown in equation 1. By repeating the above procedure for any given frequency, the surface wave velocity corresponding to a given wavelength is evaluated, and the dispersion curve is determined. The phase data is typically viewed on the PC data acquisition system in the field to ensure that acceptable data is being collected. The phase data is then returned to the office for further processing. The phase of the cross power spectrum (transfer function) between the two receivers and the coherence function are used in creating the dispersion curves.

After frequency masking of the phase record pair from the data set for each test location, an experimental field dispersion curve is developed that is the plot of surface wave velocity versus wavelength. Experimental dispersion curve data is presented in Figure 3. from a test at a sound location on a dam face. This location showed no evidence of surface degradation from a visual inspection, and also showed no evidence of degradation in the SASW data results. Thus, this plot shows the relatively flat, high velocity (6,500 feet per second, fps) dispersion curve typical of sound concrete.

A second sample data plot is presented in Figure 4. This is a plot from a test point farther down the same dam face, where the surface appeared to be distressed from freeze-thaw damage. The degradation of the concrete at the surface is supported by the SASW test results, which also provide an estimate as to the depth of degradation

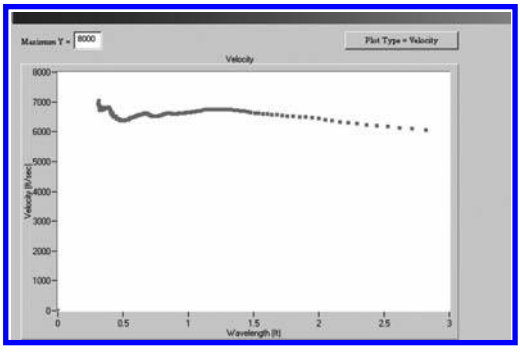


Figure 3. Sample SASW dispersion curve—sound concrete.

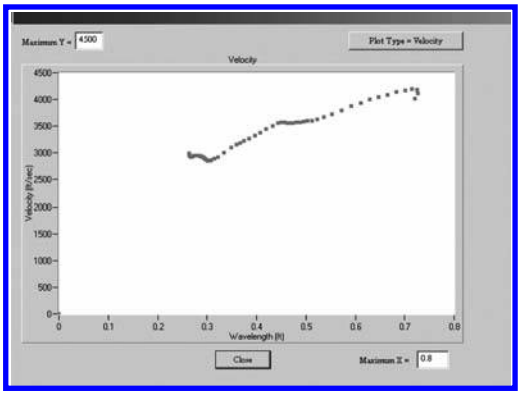


Figure 4. Sample SASW dispersion curve—freez-thaw crack damage depth to 0.3 feet in concrete.

into the concrete face. The degradation is evidenced by the low initial velocity of the plot (3,000 fps) which then begins to increase at a wavelength of about 0.3 feet. This is typical of concrete with significant surface degradation out to a depth of about 0.3 feet, at which point the sound concrete at deeper depths begins to raise the average surface wave velocity for longer wavelengths.

4 GROUND PENETRATING RADAR (GPR) METHOD

The GPR method involves moving an antenna across a test surface while periodically pulsing the antenna and recording the received echoes, as diagramed in Figure 5 for a typical concrete slab. Pulses are sent out from the GPR computer driving the antenna at a frequency range centered on the design center frequency of the antenna, in this case 400 MegaHertz (MHZ). These electromagnetic wave pulses propagate through the

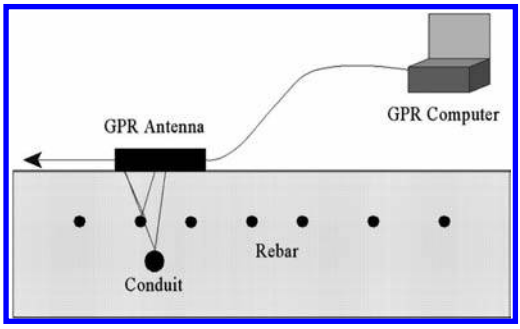


Figure 5. GPR method.

material directly under the antenna, with some energy reflecting back whenever the wave encounters a change in electrical impedance, such as at a rebar or other steel embedment or air-filled void. The antenna then receives these echoes, which are amplified and filtered in the GPR computer, and then digitized and stored. A distance wheel records scan distance across the test surface and embedded features can be located as a given distance from the scan start position. For repetitive scanning, a standard survey is designed and adhered to as field conditions allow to minimize mistakes and to maximize data quality. Daniels expounds on the theory and application of the GPR method (1996).

The scans for this investigation were created from pulses sent out at lateral intervals of near 48 pulses per foot. The resulting raw data is in the form of echo amplitude versus time. By inputting the dielectric constant (based on the material being scanned; 9.8 was used for this investigation), and by estimating the signal zero point, the echo time data can be converted to echo depth. If more accurate depth data is required, a depth calibration can be done if an embedment of a known depth is available to scan over. Concrete electromagnetic velocity (dielectric constant) calibration was performed using 8 spillway thickness values obtained from the cores taken. The scans are then typically plotted as waterfall plots of all of the individual data traces collected, with the lightness or darkness (or color) of each point in the plot being set by the amplitude and polarity (positive or negative) of the data at a given depth in each trace. Further, if data are collected along evenly spaced gridlines, a 3-D interpolation can be performed to generate a cubic display of data. This data cube can then be sliced along certain planes (typically XY, XZ, and YZ) to enhance recognition and display of target features. Also, amplitude threshold constraints can be set to allow display of GPR reflections within the given threshold values. Regional features are often more easily recognized when viewing a slice of 3-D interpolated data.

4.1 *GPR field investigation and example data*

To simplify field data collection for GPR and SIR, a nominally 4 × 4 ft grid was established at the spillway. A naming convention was used to designate each grid line in both the north-to-south and east-to-west directions. The centerline of the spillway, running longitudinally for more than 156 ft downstream (north), was named 'C'. Longitudinal lines were designated at 4 ft intervals from right of center eastward as R1 through R6, and at 4 ft intervals from left of center westward as L1 through L6 when looking downstream from the spillway crest. The short east-to-west gridlines (axial lines) began with Line 1 located 2 ft downstream of the reservoir shore edge and continued at 4 ft intervals to line 40, approximately 158 ft downstream of the reservoir shore edge.

The GPR investigation was performed over a nominal length of 156 ft along the concrete spillway. All GPR data were collected using an antenna with a center frequency of 400 MHz and stored in a Geophysical Survey Systems Incorporated (GSSI) SIR-2000 GPR field data collection system. The GPR scanning was performed in 13 lines along the length of the spillway at lines spaced at 4 ft intervals. The actual scanning was split into 2 portions–upper and lower spillway. Initially, scanning was performed at the upper portion of the spillway, from the crest downstream to a distance of 120 ft (Line 30). Next, the lower spillway was scanned, from 116 ft (Line 30) to near 156 ft (Line 41) below the spillway crest. All GPR scans were started 2 ft from the reservoir shore edge and continued downstream. The spillway tapered down uniformly from upstream to downstream causing the two outside lines on both sides of the spillway (L6, L5, R5, and R6) to 'pinch out' at a distance less than 156 ft. The two outside scans on both sides of the spillway are shorter than the nine inside scans.

An example of GPR data recorded along the spillway center is presented in Figure 6. This is a waterfall plot of GPR data recorded along the centerline, 70–107 ft downstream of the spillway crest. The waterfall plot presented in the figure was manipulated and analyzed using a 2-D GPR analysis and display software package. Figure 6 shows 3 scan characteristics. The rebar mat ranges from a depth of 0.2 to 0.6 ft (depth in ft. on right hand vertical scale—left hand scale is time in nanoseconds). The slab bottom reflection is evident at a depth ranging from approximately 0.6 to 0.8 ft. The amplitude of the slab bottom reflection was used for void detection analysis with "bright" and "low" amplitude reflectors shown in Figure 6.

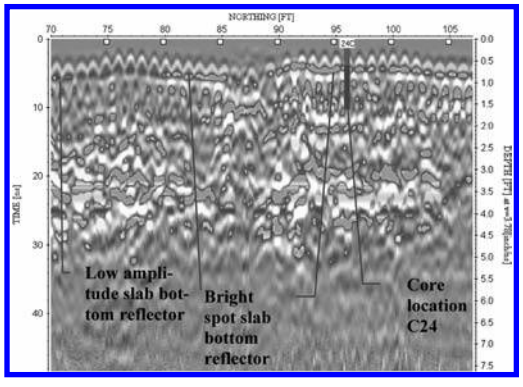


Figure 6. Example GPR data plot from the centerline, 70–107 ft below spillway crest.

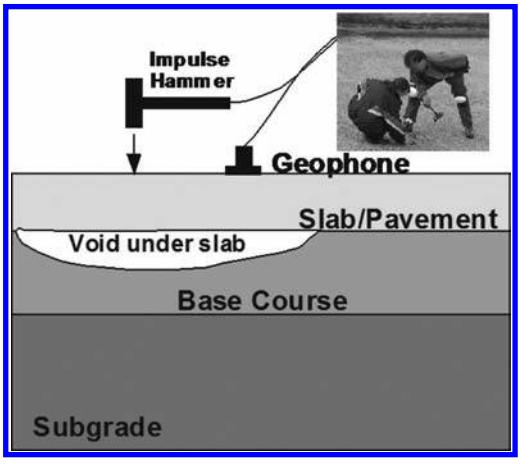


Figure 7. Slab Impulse Response method (SIR).

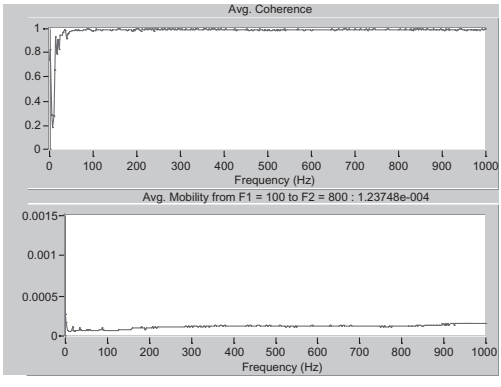


Figure 8. SIR mobility for good subgrade support.

Stronger negative amplitudes are indicative of a strong dielectric constant contrast (concrete to air), evidence of potential voids. The core location along the centerline, used for concrete dielectric constant calibration, is shown in the figure as well (near 96 ft). The concrete at this location measured 7.25 inches thick.

5 SLAB IMPULSE RESPONSE (SIR) METHOD

The Slab Impulse Response (SIR) method detects and defines the extent of good versus void/poor support conditions of a slab, but does not provide information on the depth or thickness of void (Figure 7). The method was developed from a force-response vibration test for investigating the integrity of deep foundations and was originally adapted for a slab by a European group. The SIR investigation was conducted from the surface of the concrete spillway. Field equipment included an impulse hammer, Wilcoxon velocity transducer, and an Olson Instruments Freedom PC. The method involved hitting the concrete spillway to generate vibration energy. The 3-lb impulse hammer has a built-in load cell with a plastic head to measure the force of the impact. The vibration response of the concrete to the impact is measured with the velocity transducer held in contact with the concrete close to the point of impact. The outputs from 3 hits of the fhammer and the receiver responses were viewed and recorded in the field and processed in the office.

The SIR data is processed on the PC with Fast Fourier Transform (FFT) Transfer Function operations on the time domain data to produce the mobility plots in frequency domain. Figure 8 is an example plot of mobility (vibration velocity amplitude per pound force) as a function of frequency measured in cycles per second or Hertz (Hz) for good subgrade support (bottom plot). The low, and comparatively smooth, mobility is an indicator of good subgrade support conditions. Irregular and higher amplitude mobility indicates a less stiff slab-subgrade support system, indicating poor (void) support conditions (Figure 9). The top plots in Figures 8 and 9 are of coherence and a value near 1.0 indicates good quality data.

Subgrade support condition evaluation is based on several measured parameters. First, the mean mobility (in/sec/lbf) provides a general indication of the spillway stiffness. Higher mobility may indicate a more flexible and less stiff spillway-subgrade system. Secondly, the shape of the mobility plot at frequencies above the initial straight-line portion of the curve (between 100 to 800 Hz in this investigation) is another indicator of subgrade support conditions. The response curve is more irregular and has a greater mobility for void versus good support conditions due to the decreased

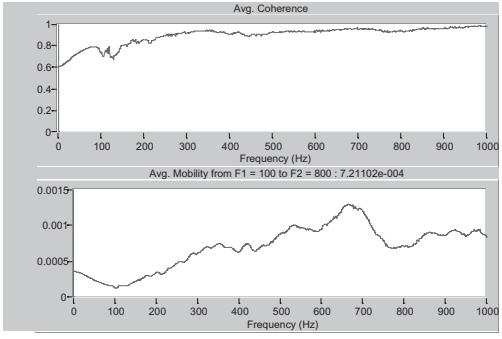


Figure 9. Example SIR plot of mobility indicative of void/poor subgrade support.

damping of the spillway vibration response for a void (Figure 9). Finally, the initial slope of the mobility plot gives the low-strain flexibility (in/lbf) of the spillway-subgrade system. The flexibility is a measurement of how much the spillway moves for a given impact, and the inverse of the flexibility is stiffness. Higher flexibility corresponds to less subgrade support or thinner concrete at the data point. Additional discussion of the SIR method and its history is given by Davis (1999).

Other factors typically considered in the SIR method include the geometry and thickness of the spillway, the boundary conditions in the vicinity of a test location (including cracks and joints), and the spillway reinforcement. Findings and conclusions on the spillway subgrade support conditions can usually be drawn based on SIR results, comparison of data from similar conditions, and/or by correlation with destructive (e.g., core) results. With other factors being constant, thinner spillways are more mobile and flexible than thicker spillways. Regardless of the thickness, the shape of a mobility curve from a point with good subgrade support is generally smooth with no low frequency peaks.

5.1 *SIR field investigation*

For the collection of the SIR data, 428 data points were taken along the spillway on a grid spaced at 4 ft intervals along the 156 ft tested length of the spillway, and over the entire width of the spillway. A nominally 4 × 4 ft grid was established at the spillway as described in the previous section and repeated herewith. A naming convention was used to designate each line in both directions. The centerline of the spillway, running longitudinally for more than 156 ft downstream, was named 'C'. Longitudinal lines were designated at 4 ft intervals from right of center eastward as R1 through R6, and at 4 ft intervals from left of center westward as L1 through L6 when looking downstream from the spillway crest. The short east-west trending lines (axial lines) began with Line 1 located 2 ft downstream of the reservoir shore edge and continued at 4 ft intervals to line 40, approximately 158 ft downstream of the reservoir shore edge.

This grid system allowed for the generation of an image contour map that relates each location's mobility to the corresponding mean values of all SIR data collected at the site in a relative sense (Figure 10). This image map was created from analyses of mobility and coherence plots.

5.2 *Discussion of GPR and SIR NDE results*

The results of the NDE investigation are presented graphically in Figure 10 which presents plan and perspective views of 3-D GPR data showing strong amplitude slab bottom reflection values, an image map contouring the SIR relative mobility values, and the core locations performed after the NDE investigation. The SIR and GPR results are discussed below.

5.2.1 *GPR results*

After GPR data collection, the raw data were post-processed in our office to enhance target features and remove background and ambient noise. The digital processing steps for each scan on the upper portion of the spillway included 1) trace zeroing and 2) background noise removal. For the lower portion of the spillway, GPR data also underwent an automatic gain algorithm to normalize trace amplitude from scan to scan. The GPR data

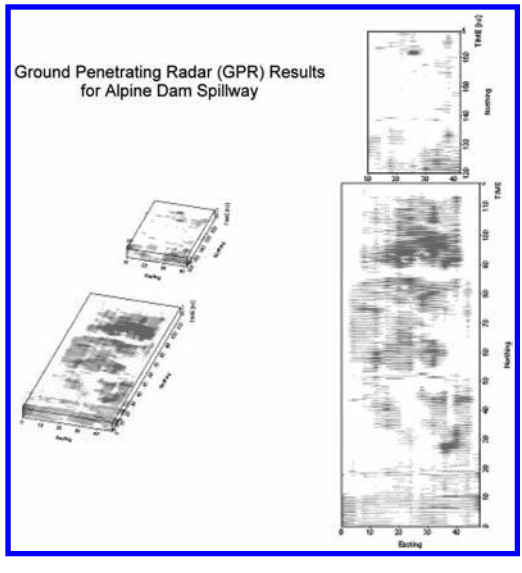


Figure 10. GPR results for spillway slab subgrade void areas indicated by brighter, darker GPR reflections.

presented in Figure 10 (in both perspective and plan views) show the "bright", strong negative values for slab bottom reflection indicative of void and are the result of 3-D interpolation between the 13 lines scanned. The plots are separated into 2 sections, the upper and lower portions of the spillway. Areas of strong negative amplitude slab-bottom reflection or "bright spots" (red and orange shading in Figure 10) were interpreted as areas of potential voids. These bright spots are evidence of the strong contrast between the electrical properties of concrete and that of water or air-filled void versus the weaker contrast between the electrical properties of concrete and subgrade soil. From the GPR data, much of the spillway shows evidence of potential voids. The two largest areas are between 55–80 and 90–110 ft below the spillway crest. Smaller areas of potential void appear in Figure 10a as well. The GPR data did not allow for potential void thickness/depth approximation because of poor void bottom resolution; the core results provided the type of data. Core thicknesses were instrumental in our ability to identify the spillway thickness and hence identify areas of void.

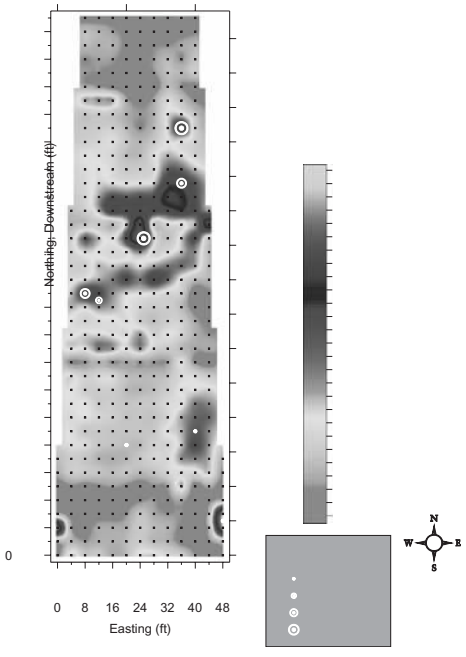


Figure 11. SIR relative mobility plot and core location void thickness results.

5.2.2 *SIR and coring results*

The Slab Impulse Response (SIR) evaluation method is fundamentally used for the identification of potential shallow to deeper voids located within or directly below a concrete slab. The SIR method cannot identify the actual depth or thickness of a possible void, but can find the plan view location along an investigated portion of spillway in which a void might exist. From the SIR data performed on the spillway, an image contour map was created (Figure 11) relating the mobility of all data points to the spillway mean mobility value. These plotted values have been normalized to the mean mobility value recorded at the spillway. This image contour map in Figure 11 presents several sizeable areas for which the mobility is higher than the mean value for the spillway (bright yellow, red, and blue shades). The two largest and potentially most severe areas of void are between 55–80 and 90–110 ft below the spillway crest (red and blue shades). Based solely on the SIR, these areas may indicate potential void beneath the spillway or may reflect a change in slab thickness. The coring results indicate that concrete thickness varies considerably over the length of the spillway. Comparisons of core data with mean mobility values confirmed small to large voids at SIR locations with normalized mobilities ranging from 0.6 and higher. Consequently, all SIR data points with normalized mobilities greater than 0.6 are likely to have poor void support conditions. Even lower mobilities may have questionable support as no core encountered good support conditions.

5.2.3 *Correlation of GPR and SIR results*

The GPR and SIR data correlate well as is evident from the plots presented in Figures 10 and 11. White areas in the GPR plots (weak amplitude slab bottom reflection) correspond to green areas (low mobility) in the SIR image map and suggest locations for better subgrade support. The coring results, Table I, support the NDE data set results. Figure 11 plots the core locations and the size of each thickness varies considerably over the length of the spillway. Comparisons of core data with mean mobility values confirmed small to large voids at Slab IR locations with normalized mobilities ranging from 0.6 and higher. Consequently, all Slab IR data points with normalized mobilities greater than 0.6 are likely to have poor void support conditions. Even lower mobilities may have questionable support as no core encountered good support conditions.

5.2.4 *Correlation of GPR, SIR and core results*

The GPR and Slab IR data correlate well as is evident from the plots presented in Figures 10 and 11. White areas in the GPR plots (weak amplitude slab bottom reflection) correspond to green areas (low mobility) in the Slab IR image map and suggest locations with better subgrade support. The coring results support the NDE data set results. Figure 11 plots the core locations and the size of each symbol

Figure 12. Borescope photo of slab subgrade void.

relates to the thickness of the corresponding void. Voids were encountered for all locations cored, however, the thinner voids were found in areas of relatively low mobility (Slab IR) and weak slab bottom reflector (GPR). One exception was a core where a thin, 0.5 inch thick void was discovered. A proper explanation of this core may be the fact that significant voids are often closely surrounded by areas of good subgrade support, within a 3 to 5 foot radius. Even with NDE results to guide core location selection, coring can be a hit or miss operation. The correlated results infer that the Slab IR image map does indeed show locations of potential voids as opposed to changes in slab thickness.

Video borescope probing was performed in the coreholes to provide visual confirmation of the NDE results. Both individual JPEG images as well as VHS recordings were obtained in each corehole. Figure 12 shows a borescope still-shot from corehole 9 L1 (Easting line L1, Northing line 9—see Figure 11 for location). The spillway concrete, void, and underlying subgrade are all evident in the still-shot. The images recorded with the video borescope showed voids extending from the coreholes and proved the NDE results for all coreholes tested.

The combination of GPR and Slab IR was proven to be successful in delineating subgrade voids at the alpine dam spillway. The combined methods also provide an accurate, fast, nondestructive, and inexpensive way to provide both qualitative and quantitative data regarding void location, size, and extent. The coring and corehole borescope investigation verified the accuracy of the NDE methods.

6 ACOUSTIC TOMOGRAPHY (AT) TEST METHOD AND EXAMPLE RESULTS

The Acoustic Tomography (AT) test method is used to provide information on the interior structure and condition of concrete elements where access is available on two sides of the element. Sound wave energy is sent from a number of evenly-spaced source locations on one face to a number of evenly spaced receiver locations on the opposite face (Billington et al, 2001). With enough source-receiver combinations, the resulting data set can be processed with a tomographic processing software package and a velocity contour image of the interior of the element created.

The application of the method to thin-arch concrete dams has been refined to improve data collection rates by using a hydrophone string suspended in the water on the upstream face as a receiver array. This allows many ray paths of data to be collected from each source impact location. With enough receivers, the full tomographic data set can be collected in a single top-bottom pass of the sound wave source. The full set of data paths, or rays, is ideally a dense grid of overlapping lines such as the ray path plot shown in Figure 13. This

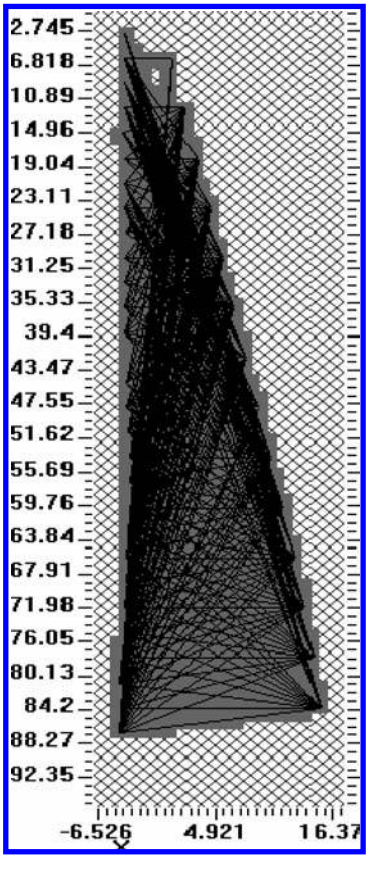


Figure 13. Typical AT raypath plot showing ray density on dam cross-section (distances in ft).

particular example used 300–400 ray paths to generate the resulting tomogram.

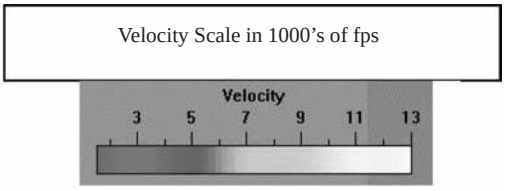
Typical sound wave sources for AT tests on dams have included instrumented hammers used by rope-suspended climbers, or electrically-operated solenoid impactors mounted on a rolling frame lowered down the downstream face. In a recent investigation, a technician on a rope used a 3 pound instrumented modal impulse hammer to impact the rough, freeze-thaw damaged concrete dam face. The impacts from the hammer allowed the input of high-amplitude signals which could be picked up by the hydrophone string even at high source to receiver angles and distances and in areas with poor surface conditions.

6.1 *Example AT data*

The data collected from AT field tests are analyzed to pick the compression wave arrival times. The compression wave travel times and geometry are then input into a tomography calculation software package which computes the velocities in the interior of the dam concrete based on all of the signals which pass through each interior point. The computed velocity data is then input into an advanced modeling software to produce velocity tomogram images such as the one presented in Figure 14.

Typically, tomographic images are analyzed to look at the velocity changes within the concrete. Areas with lower velocity correspond to weaker, less dense concrete, while those with higher velocities are considered to be sound concrete. The results can also show areas with cracking damage or other discontinuities in concrete. It should be noted that in tomographic analysis, it is also possible to get what are called "artifacts" in the data in areas with low ray density (not many data paths passing through an area) or in areas with non-linear or non-homogeneous velocities. The example tomogram presented in Figure 14 includes a few of these artifacts at or near the downstream face. The artifacts were most severe near the top of the dam where the ray density was very low. Close examination of the data showed that most of these zones are localized right at the underwater hydrophone receiver locations, and are likely due to the effects of variable coupling between the receivers and the concrete through a rubberized membrane and drainage fabric layer which were in place on the upstream face at the time of testing.

The AT test results shown in Figure 14 provide a picture of the interior condition of the concrete through a slice of the dam from the downstream to upstream faces. These results show generally sound concrete through the dam cross section, with only minor downstream face surface degradation, down to about elevation 85. Below an elevation of



Note: Low velocity zones correspond to /degraded concrete areas-white areas are untested.

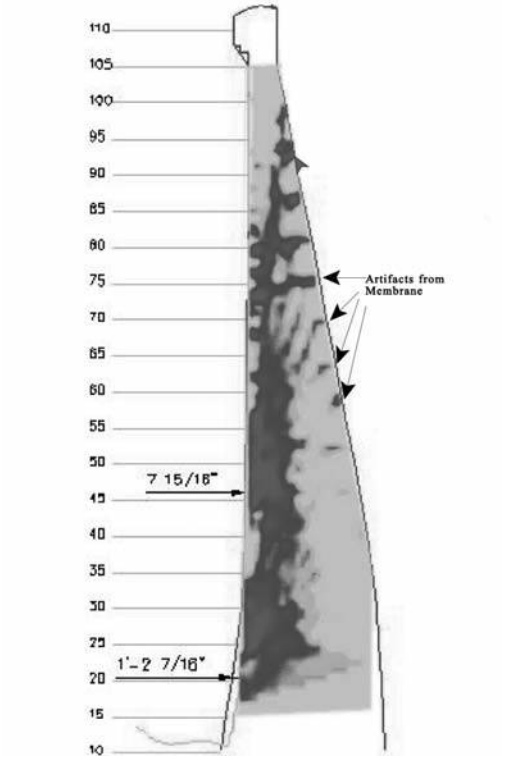


Figure 14. AT velocity tomogram-upstream right.

85 feet, the degradation becomes more uniform, with the AT results showing degraded concrete out to a depth of about 5–7 feet or more from the downstream face. As seen in the figure, the most severe areas of degradation from the AT results are at elevations 22–34 and 44–57.

7 IMPACT ECHO (IE) DISCUSSION

Impact Echo (IE) has also been used on dams with thicknesses of 10 ft or less to define the extent of cracking and concrete quality (Olson et al. 1994). This one-sided access, resonance based test is based

on impacting the concrete and recording resonant echoes to indicate the thickness of elements as well as internal flaws such as void and cracking in concrete.

8 CONCLUSIONS ON CONCRETE DAM NDE

Several methods for NDE investigations of concrete dam elements have been presented, each of which can provide information on damage and degradation not available from a simple visual inspection. The methods presented include SASW tests for surface damage and crack depth, GPR for rebar location and GPR and SIR for spillway slab subgrade void location, and AT tests to survey the entire cross-section of a dam interior and reference to IE tests as well.

Spectral Analysis of Surface Waves (SASW) testing proves to be most useful in mapping the lateral extent and the depth of degradation that is often associated with freeze-thaw cracking damage. SASW does not require dewatering of the dam or significant preparation of the dam face. The results from this testing can greatly benefit the owners and operators by identifying minor degradation before it becomes a major problem.

Ground Penetrating Radar (GPR), because it is an electromagnetic method, determines the location of steel reinforcement and can provide insight on the lateral extent of subgrade voids. The location of reinforcement is a crucial exercise for retrofitting a spillway. Identifying the subgrade voids at an early stage will provide the owners and operators with the information necessary to solve minor erosion problems before they become serious issues.

The combination of GPR and Slab Impulse Response (SIR) NDE was proven to be successful in delineating thin voids of less than 0.5 inch (SIR only) to thicker subgrade voids (SIR and GPR) at the alpine dam spillway. The SIR and GPR methods also provide a fast, nondestructive, and inexpensive way to provide both qualitative and quantitative data regarding void location, size, and extent. The coring and corehole borescope investigation verified the accuracy of the NDE methods. The NDE methods are capable of replacing coring as an exploratory technique. However, coring should be performed for confirmation of NDE to validate the data and to build confidence in the general public in the use of remote sensing.

Acoustic Tomography (AT) testing is used to analyze the cross-section of the dam providing insight to the low velocity or low strength areas that may have been visually identified. These areas are usually recognized from visual inspection. However, the extent of low strength areas is generally unknown until AT testing is performed. The information gained through AT testing will help the owners and operators determine weak areas and assess the expected life of the dam. Impact Echo has also been found to be useful for cracking/void/thickness NDE of concrete dam elements up to 10 ft in thickness.

The methods presented are all completely nondestructive and non-invasive, and usually can be applied to dams without dewatering or other interference with normal operations. Together, these methods provide a powerful set of tools that can be used by owners and operators of dams and similar facilities to ensure that the structures in their care continue to safely perform the service for which they were originally constructed. Each of these technologies provides insight to individual problems recognized by visual inspection. When NDE methods are combined and analyzed as appropriate for a given dam project, they provide the owner and operator with the information essential for assessing the dam's longevity and structural safety.

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System of thresholds of the Valle Grande dam instruments

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ABSTRACT: In the present work the probabilistic procedure used for the determination of the system of thresholds of the Valle Grande dam is described, indicating limits or ranks of operation of normality as well as of anomalies of a series of detectors instruments, which will require in case of being surpassed an immediate analysis to determine the existence of risks for people and dam. From the analysis of the registries of the system of pendulums, a differentiated behavior is observed when the water lever exceeds level 800, with a greater deformation of practically all the modules in upstream—downstream direction and a more marked movement towards the valley when this level is surpassed. As well, a feeding mechanism of volumes is observed in the abutments, characterized by sudden increases that occur as much in the months of colder temperatures, like when the reservoir surpasses the level ≈ 800. Thanks to this singular characteristic, the proposal of a system of thresholds, for the Valle Grande dam, requires of the study of the behavior of the detectors instruments in the two situations enunciated. Consequently two families of thresholds obtain themselves, one for dam levels superiors to the level 800 and another one for levels inferiors to this level.

1 INTRODUCTION

The Valle Grande dam is located on the Atuel river, in the province of Mendoza, the region central west of Argentina. It is one of the provinces that compose the region of Cuyo, at the foot of the Andes, ranging between 32° and 37° 35' south latitude and 66° 30' and 70° 35' west longitude.

Los Nihuiles hydroelectric system is the system of hydroelectric generation and regulation of the Atuel river. It is formed by four dams: El Nihuil, Aisol, Tierras Blancas and Valle Grande and four power stations: Nihuil I, II, III and IV. The regulation of water for irrigation and for human consumption is through the Valle Grande dam, located 35 km upstream from the city of San Rafael, which has a stable population of 180,000 inhabitants.

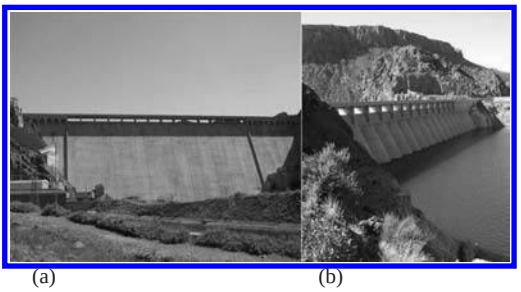

Figure 1. a) Downstream view of the dam; b) Upstream view of the dam.

This 46 years old dam, is located in Zone 3 of the zoning established in the INPRES—CIRSOC 103 that corresponds to "high seismic hazard," since that is installed in the contact zone between continental and oceanic tectonic plates of great instability that make a band of high seismic risk.

The Valle Grande dam is a round head buttress dam and is founded directly on rock, with a peak altitude between rock and the top of dam of 115 m. It owns 24 buttresses in double-T, with round concrete heads Noetzli-Alcorta type. This type of profile presents the same width in the nose that in the pillar. The crest length is of 310 m and the spillway is of Kreager profile of 14 openings, with a capacity of evacuation of 600 m^3/s.

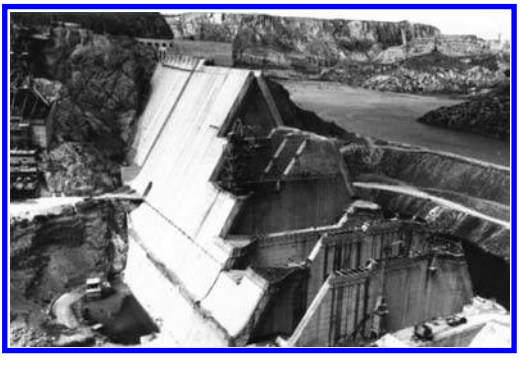

Figure 2. Historical photo during construction of the Valle Grande dam—year 1964.

The safety of a dam depends very critically on their correct observation to see if it behaves as intended or if they have abnormal behavior. It is very rare that a dam to break unexpectedly, before reaching a critical situation seen a number of symptoms: large increase in leakage, abnormal deformation, cracks, etc. So it is so important systematic observation and interpretation of data.

The auscultation system of the Valle Grande dam is designed to control two parameters of singular importance: the deformations of the dam and seepage from contact with the abutments or from the body of the structure.

The deformations are controlled from two complementary and independent systems: a system of pendulums and the microgeodetic control of the movements of points located in the gallery of level 805, from the collimation of these points in reference to independent fixed points located in both abutments. The thresholds only are applied to the permanent reference instruments (detectors) not to those of verification or redundancy (microgeotetic control) which has smaller measurement frequencies.

In the Valle Grande dam, the detector instruments are pendulum and flow gauging:

- Pendulums: the deformations of the dam is controlled through a system of pendulums, 4 of which are direct and 4 are inverted, distributed in six lines.
- Flow Gauging: the system of drainage of the Valle Grande dam is made up of 24 flow gauging stations, of which 21 flow-meters are located within the body of the dam and 3 are placed downstream on both abutments. This drainage system's main purpose is to relieve potential pressure in the contact rock-foundation of the different modules of the dam. In the basement of the central sector it is made up of numerous relief wells of great diameter (2 m) that reach the contact with the rock. In the abutments however, the volumes collected by the system of contact drains are brought through the galleries located at the base of the modules. In this point they are measured in order to keep under observation its evolution over time.

2 THRESHOLDS

A Threshold marks a zone from which the behavior of the detector device does not respond as it is hoped. In a thresholds system a variation rank is established, the measurements that are outside the same correspond to a singularity of measurement or an outstanding anomaly.

The thresholds are used like a control tool of the detector devices data. It is not tried this way to automate the monitoring system of the dam, but offer to the operator an additional attendance to detect possible anomalous behaviors in the structure.

2.1 *Reference levels*

Two reference levels associated to apartments of different order in the habitual behavior are considered:

- "Singularity of Measurement" (S.M.), it is an apartment of reduced magnitude with respect to the habitual behavior, without consequences on the security of the dam. This threshold determines the attention on the behavior of certain device, requiring itself the necessity of an immediate verification of the measured values.
- "Outstanding Anomaly" (O.A.), it is a greater discrepancy, that it can mean a particular behavior, possibly dangerous of the dam. This way it is tried to bring about a deep reflection of the auscultation personnel on the behavior of the dam or its instrumentation. If the problem cannot be clarified it is necessary to take measures that would have to be indicated in the Action Plan During Emergencies (PADE).

2.2 *Types of thresholds*

Based on the type of parameter and its evolution throughout the last years, the following thresholds types are used:

- "Fixed Thresholds": they are used in all those devices whose behavior cannot be fit in statistical form based on the evolution of the level of the lake or the temperature. Generally they are established from maximum or minimum values in the recent history of answer.
- "Configurable Thresholds": established for those devices whose history of registries can be fit reasonably by statistical procedures based on the variations in the reservoir level, the temperature, etc.

3 STATISTICAL MODEL

The calculation of expected values of reading's main purpose is to monitor the functioning of the instruments and the detection of unusual phenomena.

Since the values registered by the instruments depend on the state of loads, the estimation of the data can be made from knowledge of the value of the loads and by applying empirical or deterministic methods.

The use of deterministic methods is not always feasible given both the complexity of the processes

involved and the ignorance of the values of the parameters that characterize the system.

The application of an empirical method is to establish the relationship between cause and effect based on the observation of phenomena. The adjustment of the obtained results will depend on the way in which these relations are defined.

For the determination of the expected value of the data of an instrument an empirical model has been implemented where the function that establishes the relation between the causes and the effects is of the form of equation (1):

$$EV = C_1 + C_2 \cdot Z + C_3 \cdot Z^2 + C_4 \cdot Z^3 + C_5 \cdot R + C_6 \cdot R^2 + C_7 \cdot R^3 + C_8 \cdot sen(\omega t) + C_8 \cdot \cos(\omega t) \quad (1)$$

were:

- EV = expected value
- Z = (LL max-LL)/(LL max-LL min)
- R = (ARL max-ARL)/(ARL max-ARL min)
- ω = frequency of the harmonica of annual period.
- t = day number in the year (1 a 365)
- LL = lake level
- LL max = maximum lake level
- LL min = minimum lake level
- ARL = afterbay reservoir level
- ARL max = maximum afterbay reservoir level
- ARL min = minimum afterbay reservoir level.

The constants C1 to C9 are calculated by means of a multiple linear regression based on minimum squares. In those instruments that has an evolution of the irreversible type (a permanent variation of its annual average, independent of the reversible variations generated by the reservoir, temperature, etc.), has been included an additional term that would consider an evolution of linear type in the time, valid per periods limited and are fitted in periodic form.

As a result of the application of multiple linear correlation values obtained from 9 constants, the correlation coefficient, the standard deviation of the residues and partial correlation coefficients for each term. The correlation coefficient indicates the precision of the adjustment, the standard deviation, the size of apartments of the expected value once adopted the probability of occurrence (the probability that the value recorded by the instrument fall within EV ± standard deviation is 70%) and partial correlation coefficients indicate the influence of each term over the total adjustment (highest value of partial correlation coefficient indicates greater influence of the term).

Once calculated the Expected Value the following variables are defined:

Y = measured value
σ = standard deviation of the statistical model
TOL = tolerance between Y and EV that is adopted equal to 2σ in the threshold of Singularity of Measurement and equal to 4σ in the threshold of Outstanding Anomaly.

Based on the enunciated variables it is possible to define then the thresholds superior and inferior, according to the following condition:

$$Y \leq EV \pm TOL \quad (2)$$

For the configurable thresholds it is possible to define:

$$D = \text{abs}\,(Y - EV) \leq TOL \quad (3)$$

that it is the value of comparison.

For fixed thresholds, which do not adjust to some law, the following equation is adopted for the determination of the value of the Singularity of Measurement and the Outstanding Anomaly:

– Condition so that the EV is a S.M.

$$EV \geq FL_{SM} \quad (4)$$

were FL_{SM} is the fixed limit that has been determined for a Singularity of Measurement, therefore we are outside the rank of the awaited value when

$$\left| \frac{EV}{FL_{SM}} \right| \geq 1 \quad (5)$$

– Condition so that the EV is an O.A.

$$EV \geq FL_{OA} \quad (6)$$

were FL_{OA} is the fixed limit that has been determined for an Outstanding Anomaly, therefore we are in O.A. rank when:

$$\left| \frac{EV}{FL_{SM}} \right| \geq \frac{FL_{SM} + \Delta FL}{FL_{SM}} = 1 + \frac{\Delta FL}{FL_{SM}} \quad (7)$$

were

$$\Delta FL = FL_{OA} - FL_{SM} \quad (8)$$

4 DIRECTIVE CRITERIA AND THRESHOLDS PROPOSED FOR THE VALLE GRANDE DAM

A scheme of monitoring is configured on basis of the detection of anomalies of different degree, by means of the immediate comparison of the measured values with those understood as normal, and determined from the establishment of thresholds for each device.

With the use of this scheme, it is tried to offer a tool able to detect in immediate form any anomaly in the auscultation data, allowing to the engineering team to make the consult to the Expert Engineer of Safety Dam to study and planning action to adopt the corrective or preventive measures that correspond to each case. It is pertinent to clarify that are not due to justify all the historical data of the instruments when determining a threshold, but they are due to represent the acceptable variations of the instrument for situations of similar load.

4.1 *Reaches of the thresholds system*

The thresholds are established on the denominated "detector devices", which are tie directly to the conditions of security of the dam. These instruments have a reading frequency that allows a continuous monitoring on their evolution. In the Valle Grande dam this set includes the Direct and Inverted Pendulums and the Flow Gauging of the contact abutment-dam, on both margins.

From the analysis of the registries of the system of pendulums, a behavior differentiated from this large buttress dam is observed when the water lever exceeds level 800, with a greater deformation of practically all the modules in upstream—downstream direction and a more marked movement towards the valley when this level is surpassed (Figure 3).

As well, a feeding mechanism of volumes is observed in the abutments, characterized by sudden increases that occur as much in the months of colder temperatures, like when the reservoir surpasses the level ≈ 800.

Thanks to this singular characteristic, the system of thresholds for the Valle Grande dam requires of

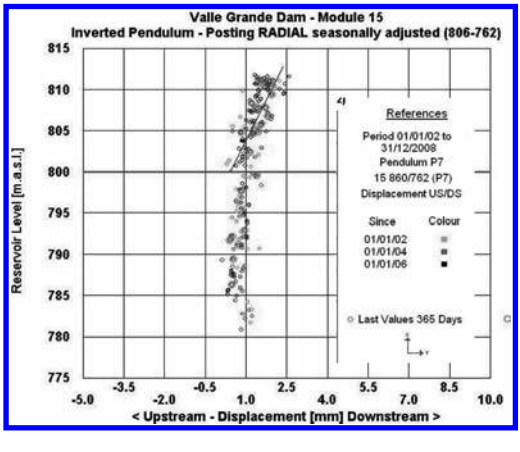

Figure 3. Pendulum 7—radial displacement versus reservoir level.

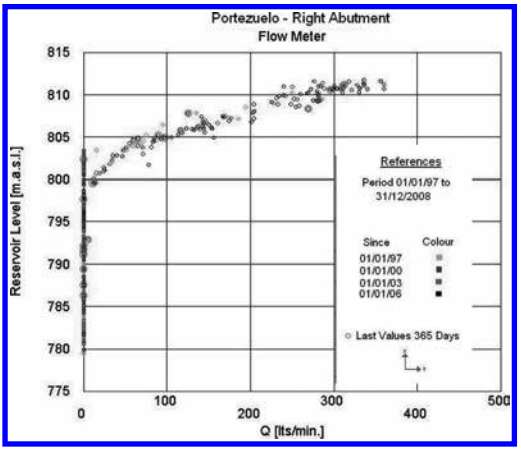

Figure 4. Portezuelo flow meter versus reservoir level.

the study of the behavior of the detectors instruments in the two situations enunciated. Consequently two families of thresholds are obtained, one for reservoir levels superiors to the level 800 and another one for levels inferiors to this limit. The graph of difference D is used in this case, unifying of this form the data that were separated based on the reservoir level.

4.2 *Establishment of thresholds for the set of detector instruments in deformation*

4.2.1 *Pendulums*

Configurable thresholds were used for these devices, associated to the Expected Value and the standard deviation of the statistical model. In Figures 5 and 6 it is presented like an example, the representation of the thresholds of a pendulum, for the two enunciated situations of water level.

Based on the criteria expressed in the point 4, it is possible to represent in a single graph the established thresholds for all possible reservoir levels. In the same the resulting differences of the Measured Value (Y) and the Expected Value (EV) are characterized as a function of the lake level, and the reference lines delimit the sectors of singular readings and the zone of outstanding anomaly that shoots an alert by the entrance of the parameter in a zone of risk.

4.2.2 *Flow Gauging*

These devices are characterized to indicate a noticeable and significant increase of the contribution of volumes when the lake surpasses the level ≈ 800. Nevertheless when the lake is below this limit, the flow is null or minimum. Based on this situation, the importance resides of dividing the models in two different states, since if the lake level surpasses

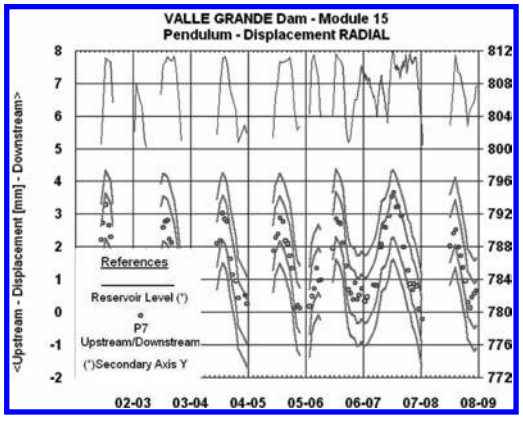

Figure 5. Pendulum 7, Radial (reservoir over 800 m.a.s.l.).

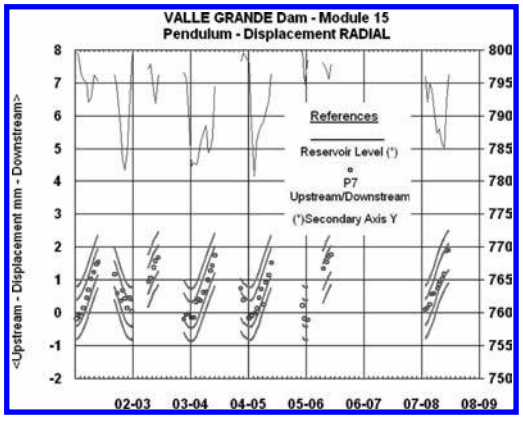

Figure 6. Pendulum 7, Radial (reservoir below 800 m.a.s.l.).

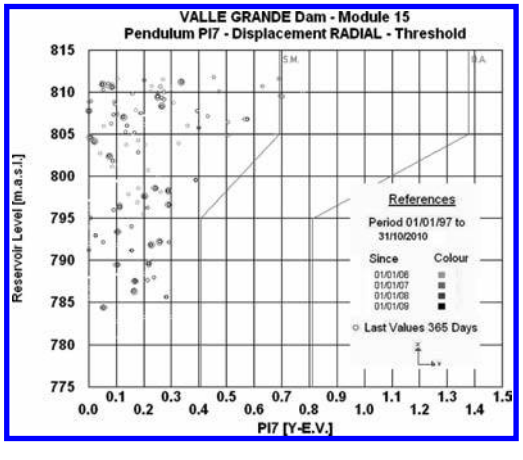

Figure 7. Pendulum 7—Radial, threshold for all possible reservoir level.

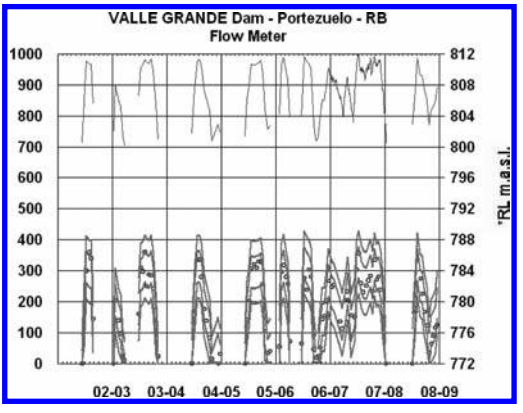

Figure 8. Portezuelo flow meter (reservoir over 800 m.a.s.l.).

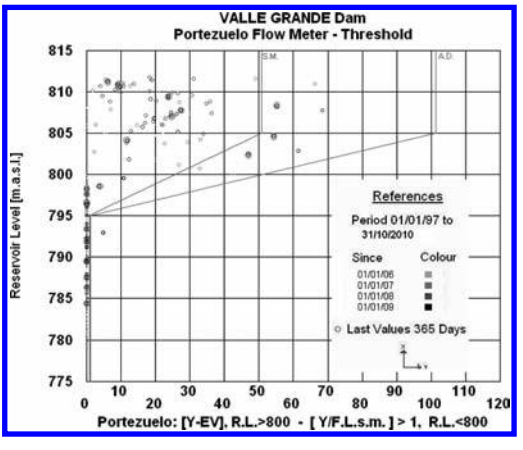

Figure 9. Portezuelo flow meter, threshold for all possible reservoir level.

the elevation 800 m.a.s.l. the volumes must be expressed within the family of thresholds.

From the statistical model, cases are observed in which the instruments present low correlation with the variables analyzed here, therefore fixed thresholds are adopted. The average of the historical registries is considered for its determination, taking like Singularity of Measurement the 10 percent from the maximums, and as well, the double to define the Outstanding Anomaly. In Figure 9 it is presented like an example, the representation of the thresholds determined for a flow-meter, for all possible reservoir Level.

5 SYNTHESIS

In the present work there has developed the calculation of the system of thresholds, which constitutes

a reference on the degree of normality of the measurements obtained of the instruments detectors and delimits a zone from which the behavior of the device detector does not answer to the awaited one.

Since the Valle Grande dam owns a behavior differentiated when the level of reservoir overcomes the level 800, as much in the referred thing to deformations like the measured volumes, in the present work the calculation of two families of thresholds for each instrument appears. One stands out that this behavior is of reversible characteristics and it is observed from the beginning of the auscultation.

The set of thresholds defined in this work will be incorporated to the auscultation software, AUSCULT made for RED Ingeniería SRL, of such form that the auscultation personnel can define in immediate form the location of the measured value in each instrument with respect to the reference levels which define the interval of normality.

ACKNOWLEDGMENTS

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Aplicación de modelos de redes neuronales artificiales a la determinación de movimientos en una presa bóveda

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ABSTRACT: The complexity of the dam-foundation makes difficult to interpret the records of auscultation. It is not easy to make a prognosis of the behavior of the dam in accordance with the different situations that may occur over the life of the dam. Nor is it easy to interpret the deviations observed from the values estimated by numerical or statistical modeling. Neural networks offer the possibility of treating the whole dam foundation as a complex system which rules of behavior are established solely on the basis of the observed behavior without making simplifying assumptions.

It has shaped the radial movement in different seasons of the pendulums available in a dam vault, which has been used as test case. We analyzed the response of the dam complex models growing. The simplest considered only as parameters determining the behavior reservoir level and a moving average of ambient temperatures (simple model). In order to model applicable to any pendulum of any dam, has raised the consideration of various means mobile temperature, so that is the model itself that determines the weight of each moving average (model general). Have also been raised as data models that incorporate the values of the movements measured immediately prior (short-term models).

The comparison of results obtained through each of the previous models of neural networks with statistical and numerical models commonly used that allows the use of neural network models useful for the interpretation of auscultation. The submission details the results obtained and the accuracy of the prognosis of each of the models studied. Models are expected to nalyze incorporate additional time variable (dynamic models), to capture the effect of drift. This research is part of the R&D financed by the Ministerio de Ciencia e Innovación: estudio de la seguridad de presas e identificación de escenarios de riesgo mediante sistemas inteligentes, reference number 048/RN08/04.5.

1 INTRODUCCIÓN Y OBJETIVOS

A la hora de emitir juicios o valoraciones sobre el estado de funcionamiento de una presa es obligado hacer, entre otras muchas cosas, un análisis por-menorizado de los resultados que ofrece la instru-mentación instalada. Este análisis, realizado con las herramientas convencionales ha resultado difícil, ya que la respuesta de la estructura presa-cimiento depende de un conjunto de variables, muchas de ellas no conocidas con profundidad, y con interrelaciones de dependencia entre ellas no sujetas a leyes pura-mente deterministas. Ello ha llevado a análisis pro-babilísticos con interpretaciones de cada uno de los parámetros basados en variables clave como el nivel de embalse o las temperaturas.

Todo este escenario se ve agravado en muchas ocasiones por la incertidumbre en la validez de los datos obtenidos bien por errores de lectura, defectos en los aparatos, etc. Además, con la interpretación tradicional es difícil modelar condicionantes como la puesta en obra, la heterogeneidad del cimiento o las características de los materiales utilizados en la construcción. Sobre estos suele existir información generada durante el proceso de control de calidad de la obra pero su interpretación depende de la habili-dad del individuo que realiza el análisis de los datos (Calarasu et al., 2008).

Las redes neuronales permiten detectar las relaciones entre los distintos parámetros involucrados en fenómenos complejos, en los que un análisis pura-mente determinista puede dejar de lado factores de indudable importancia para caracterizar el comportamiento final del problema.

Estas circunstancias han motivado que este tipo de sistemas hayan sido utilizados recientemente para tratar de comprender y analizar fenómenos tan complejos como el modelo de respuesta de cuencas hidrológicas, el transporte de sedimentos y de elementos solubles.

Diversos autores han indagado en el uso de las redes neuronales en estructuras de todo tipo. En particular, las aplicaciones a presas resultan especialmente propicias para estas metodologías por la complejidad de las variables involucradas.

En (Ahmadi and Chouinard 2003) se utilizan redes neuronales como alternativa a los métodos estadísticos tradicionales para el análisis de variables de monitorización de presas y detección de posibles anomalías en desplazamientos, tensiones y flujos. Un análisis preliminar mostró que la temperatura del hormigón es en este caso el mejor predictor de los desplazamientos de los péndulos.

En (Tayfur and Swiatek 2005), se desarrolla un modelo de redes neuronales y otro de elementos finitos para comparar los resultados con las mediciones de flujos a través de una presa de materiales sueltos. Ambos modelos fueron calibrados y verificados utilizando un piezómetro en una sección de la presa, y obtuvieron resultados satisfactorios comparados con las mediciones de niveles de agua de los piezómetros. El estudio reveló que los modelos de redes neuronales pueden ser una buena herramienta para estudiar sub-presiones. Las variables de entrada usadas fueron el nivel de aguas arriba, el nivel de aguas abajo y la identificación numérica de los piezómetros.

El modelo de elementos finitos representa el mejor entendimiento del fenómeno físico del proceso. Las relaciones entre las variables de entrada y salida están bien definidas, obteniendo mejores resultados mientras más exhaustivo sea el conocimiento de los parámetros que intervienen en el fenómeno, lo que no es fácil, dada la complejitud de las relaciones entre parámetros.

El modelo de redes neuronales es un modelo más simple, que tiene la ventaja de reconocer patrones de relaciones entre variables cuando se alimenta con una colección suficiente de datos. Desde el punto de vista práctico, el uso de redes neuronales podría ser útil para la detección de anomalías en los regímenes de filtraciones, permitiendo tomar medidas inmediatas. Sin embargo se debe destacar que las redes neuronales son una caja negra que no aportan una relación explícita entre las distintas variables. Además, las redes neuronales proporcionan resultados satisfactorios dentro del rango de datos utilizado para su entrenamiento, pero fuera del mismo el resultado no resulta fiable.

En (Wen et al., 2004), entendiendo a las presas como un sistema vital e inteligente, se propone un esquema de manejo, proceso y análisis de información en tiempo real, en el que modelos de redes neuronales y lógica difusa se usan para crear una interfaz que pueda analizar y evaluar las características de funcionamiento de las presas. Estos modelos biónicos en una primer etapa capturan la información del entorno y el interior de las presas mediante una sistema de sensores distribuidos de forma adecuada en el cuerpo de presa y sus cimientos. Toda esta información es concentrada en una unidad de procesamiento central, que analiza y busca indicios de anomalías y, si las encuentra, propone de forma dinámica contra-medidas al departamento de toma de decisiones, el que finalmente las implementa.

En (Salvaneschi et al., 1996) se destacan las ventajas de los sistemas de inteligencia artificial debido a la subjetividad involucrada en la interpretación de los datos de monitorizaciones de presas entre los cuales destaca el difícil manejo de grandes cantidades de información muchas veces incierta o incompleta, la necesidad de criterio ingenieril, conocimiento de la estructura particular, experiencia en el desarrollo de estructuras en general, y conocimientos generales de ingeniería para interpretar los datos. En (Calarasu et al., 2008), se pretende minimizar la subjetividad de los expertos humanos en la monitorización y predicción del comportamiento de las estructuras hidráulicas.

En esta investigación se pretende definir una metología de análisis de datos de auscultación mediante el uso de redes neuronales y estudiar la calidad de los resultados obtenidos en comparación con los resultados de los modelos convencionales.

2 CASO PILOTO

Se usa como caso piloto de esta investigación a la Presa de la Baells que está situada en el río Llobregat en Barcelona. Es una presa de bóveda de 3 centros y doble curvatura. Tiene una altura de 102.35 m sobre cimiento, una longitud de coronación de 403 m, un espesor en el pie de presa de 20 m y de 4 m en el arco superior. Tiene una relación cuerda/altura de 2.75 (Fig. 1).

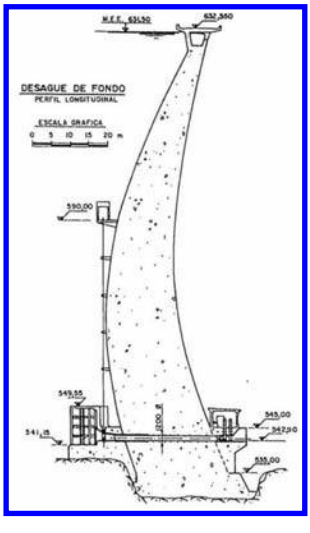

Figura 1. Presa de la Baells. Perfil.

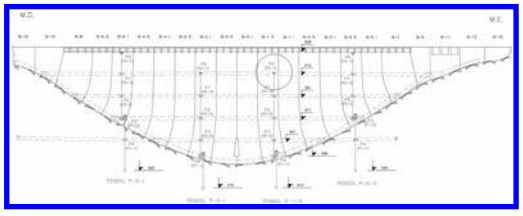

Figura 2. Esquema de los péndulos en la presa. Se señala con un círculo la cota más alta del péndulo estudiado.

Se dispone de 30 años de registros de auscultación de esta presa, distribuidos de manera regular a lo largo de su vida, lo que permite estudiar su comportamiento desde su construcción. Se ha elegido esta presa como caso piloto por la cantidad de los datos disponibles.

3 VARIABLES DE ENTRADA Y SALIDA

Se estudia el comportamiento de la presa a través de los desplazamientos radiales de los péndulos, los cuales proveen de mediciones suficientemente fiables. La presa dispone de cuatro péndulos, cada uno con registros a cinco cotas distintas. Para disminuir, en la medida de lo posible, el ruido en las mediciones se estudian los desplazamiento radiales en la cota más alta de un péndulo central. En la figura 2 se muestra la ubicación del registro estudiado.

En los análisis convencionales de los datos de auscultación se explica el comportamiento de la presa a través de variables que sean capaces de representarlo de forma suficiente. Para esto, se debe tener en cuenta la existencia de deformaciones irreversibles, el efecto del nivel de embalse y el efecto diferido del ciclo térmico anual. En este sentido, se han escogido como variables de entrada de los modelos de redes neuronales las que representan estos efectos, y que se detallan en sucesivos apartados.

4 MODELO ESTADÍSTICO

El análisis de los datos de auscultación de presas se realiza normalmente a través de modelos estadísticos. Se divide el conjunto de datos en dos grupos consecutivos, el primero se usa para ajustar una función de aproximación y el segundo para validar los resultados.

4.1 *Ciclo térmico anual*

El primer paso en estos modelos es encontrar la onda térmica anual $T(t)$, usando para ello las temperaturas medias diarias. Asumiendo un comportamiento periódico de las temperaturas se las representa en serie de Fourier. Usando la función de temperaturas se calculan las medias móviles de temperaturas de un período P, que tendrán la forma de la ecuación (1).

$$TMM_P(t) = \int_{t-P}^{t} \frac{T(s)}{P} ds \tag{1}$$

4.2 *Efecto del tiempo*

Existe un efecto irreversible en los desplazamientos de la presa debido a los movimientos reológicos que son más notorios en los primeros ciclos de la vida de la presa. Para modelar este efecto se usan funciones logarítmicas y exponenciales del tiempo.

4.3 *Efecto del nivel de embalse*

El efecto instantáneo del nivel de embalse sobre los desplazamientos se modela usando monomios de diversos grado de la cota de embalse.

4.4 *Efecto agregado de variables*

Se usan funciones de dos variables para modelar el efecto agregado de variables de distinta naturaleza como la temperatura y el nivel de embalse o el tiempo. Un ejemplo de estas funciones son las medias móviles de temperatura TMM_P que modifican la onda térmica anual en función de una inercia térmica asociada a un periodo de tiempo.

4.5 *Ajuste y comprobación de los resultados*

Se dispone de un total 1176 mediciones diarias de temperaturas, niveles de embalse y desplazamientos a los largo de 30 años. Se usa un 60% de los datos para ajustar la función de aproximación y el 40% restante para validar el ajuste.

Aunque es usual utilizar temperaturas medias móviles de 30 días (TMM_{30}) en estos modelos, la naturaleza diversa de las presas y sus particulares características hacen necesario tantear cuál es la media móvil más apropiada para cada uno de sus péndulos. En este sentido, se ha estudiado cómo varían los resultados del ajuste usando diversas medias móviles en el caso piloto.

Se define el error relativo medio de aproximar un número n de desplazamientos δ_i mediante un valor $\hat{\delta}_i$ según la ecuación 2 y los errores máximo y mínimo según las ecuaciones 3 y 4.

$$e = \frac{1}{n}\sum_{i=1}^{n}\left|\frac{\hat{\delta}_i - \delta_i}{\delta_i}\right| \tag{2}$$

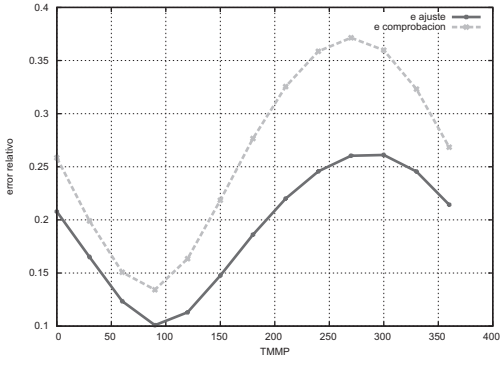

Figura 3. Variación del error relativo medio al variar el periodo de la media móvil usada para aproximar. Se grafica el error relativo medio del 60% correspondiente al conjunto de ajuste y del 40% correspondiente a la comprobación del ajuste.

Tabla 1. Indicadores de bondad de aproximación del modelo estadístico de desplazamientos para el conjunto de datos de ajuste y de validación.

Indicador	Ajuste	Validación
e	0.10	0.13
$e_{\max}$ (mm)	8.01	4.82
$e_{\min}$ (mm)	–5.78	–4.00

$$e_{\max} = \max\left\{\hat{\delta}_i - \delta_i\right\}_{i=1}^{n} \tag{3}$$

$$e_{\min} = \min\left\{\hat{\delta}_i - \delta_i\right\}_{i=1}^{n} \tag{4}$$

En la figura 3 se observa que el periodo de la tempera-tura media móvil que produce menores errores tanto en el ajuste como en la comprobación es la TMM_{90}.

Este resultado preliminar se usará más adelante en la definición de las redes neuronales.

Se apuntan en la tabla 1 los principales indicadores de la bondad del ajuste realizado.

5 MODELOS DE REDES NEURONALES ARTIFICIALES

Antes de realizar un análisis con modelos de redes neuronales, es necesario determinar si la información disponible es suficiente para comprender el fenómeno. En este caso, la existencia de un modelo estadístico con buenos resultados permite asumir que así es.

Es necesario apuntar que se debe evitar interpolar las mediciones de la variable respuesta (desplaza-mientos) pues, a priori, se desconoce lo sensible que puede ser a pequeñas variaciones de las demás variables. Respecto a los niveles de embalse, salvo casos excepcionales, suelen variar de forma lenta en el tiempo, por lo que interpolaciones lineales pueden ser apropiadas. Las temperaturas medias diarias, aunque oscilan en torno de la onda térmica anual, contienen abundante ruido intradiario, por lo que no resulta conveniente interpolarlas. En este sentido, es preferible interpolar directamente desde la ecuación de la onda térmica anual.

Para comparar los resultados de los modelos de redes neuronales con los estadísticos se usan los mismos datos de entrada y se dividen en los mismos dos conjuntos, uno de entrenamiento, correspondiente al primer 60% y otro, de validación, correspondiente al restante 40%.

5.1 *Arquitectura de la red neuronal*

La red se estructura en tres capas: la de entrada, donde habrá tantas neuronas como variables se usen para representar el comportamiento de la presa; la de salida, en que habrá una única variable correspondiente al desplazamiento estudiado, y la oculta, cuyo número de neuronas será objeto de estudio.

5.2 *Algoritmo de aprendizaje*

Se han publicado un gran número de artículos acerca de aplicaciones de redes neuronales a la ingeniería civil. La gran mayoría de estas aplicaciones están basadas en el algoritmo Back-Propagation (BP) (Budinich 1993; Habibagahi 1998; Barai and Pandey 1995; Meireles et al. 2003) que es un tipo de apren-dizaje supervisado unidireccional, aunque también hay aplicaciones de otros modelos de redes neuronales más poderosos y eficientes (Adeli 2001).

El algoritmo BP consiste básicamente en extender la regla de aprendizaje por mínimos cuadrados medios a las arquitecturas multicapa. El error se minimiza en función de todos los pesos de la red, lo que se realiza a través de derivaciones sucesivas usando la regla de la cadena lo que exige que las funciones de transferencia sean derivables. En particular, para este estudio, las funciones de transferencia en la capa oculta son sigmoideas y la identidad para la capa de salida.

El algoritmo BP, de gran generalidad, se puede aplicar a multitud de problemas diferentes. Sin embargo su convergencia es lenta, por lo que sus resultados, aunque relativamente buenos, pueden ser sustancialmente mejorados. Además no garantiza que el error alcanzado sea un mínimo global, pudiendo quedar atrapado en un mínimo local.

Para solventar esta dificultad se ha optado por entrenar un mínimo de 100 veces cada red neuronal partiendo de pesos distintos en cada ocasión.

Las redes neuronales artificiales son susceptibles de incurrir en sobre-entrenamiento, esto es, que más allá de un determinado número de iteraciones de aprendizaje, la red comienza a mejorar la aproximación de los datos usados para enseñarla a costa de los usados para validar el modelo. Usando un símil con el aprendizaje biológico, la red memoriza los resultados sin comprender el fenómeno de fondo. Para evitar esto, los entrenamientos se han realizado calculando los errores sobre el conjunto de aprendizaje y de verificación en cada iteración. Cuando el error en el conjunto de verificación empeora, se detiene el entrenamiento. Para evitar paradas en falso se continúa con el aprendizaje hasta seis iteraciones más allá del mínimo error en el conjunto de verificación.

Se han estudiado tres modelos de redes neuronales, en los que se han usado distintas variables de entrada y cantidad de neuronas en la capa oculta. Se ha variado la complejidad de estos modelos buscando un mejor ajuste y nuevas aplicaciones.

5.3 *Modelo simple*

Es el más sencillo de los modelos estudiados. En la capa de entrada incluye únicamente el nivel de embalse y una variable representativa de la temperatura. Se ha hecho variar la cantidad de neuronas en la capa oculta según la regla $2^i, i = \overline{1,8}$ para estudiar este efecto en la bondad de ajuste del modelo.

Usando el resultado de los modelos estadísticos, en los que se estableció que usando TMM_{90} se obtienen mejores resultados que usando directamente T, se han comparado los errores relativos obtenidos. En la figura 4 se muestran estos resultados. Destaca el hecho de que para alcanzar una precisión semejante a la de los modelos estadísticos, este modelo necesita de unas 64 neuronas en la capa oculta.

En la figura 5 se muestra la aproximación obtenida mediante el modelo simple y 16 neuronas en la capa oculta.

5.4 *Modelo general*

Este modelo incorpora en la capa de entrada el nivel de embalse, la temperatura media diaria T y todas las medias móviles intra-anuales de temperatura cada 30 días TMM_{P}. Al incorporar todas las medias móviles pretende ser más flexible que el modelo simple, pues es la propia red la que pondera la importancia de las medias móviles más apropiadas a la variable, estación de lectura y presa estudiada. En la figura 6 se muestran los resultados obtenidos al usar el modelo general para aproximar los desplazamientos radiales de los péndulos 1D y 6I en las cotas 610 y 571 respectivamente.

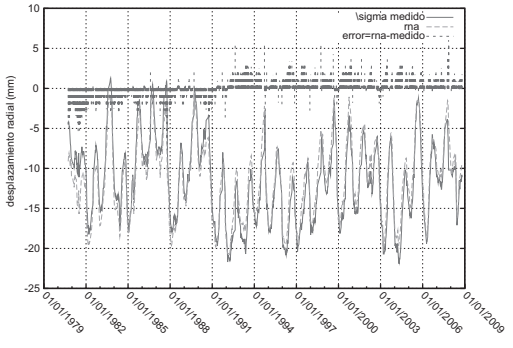

Figura 5. Desplazamiento aproximado usando el modelo simple con 16 neuronas en la capa oculta.

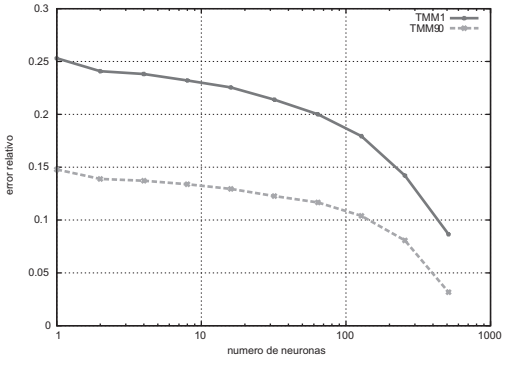

Figura 4. Variación del error de aproximación del desplazamiento usando temperaturas medias diarias T y media móvil de 90 días TMM_{90}.

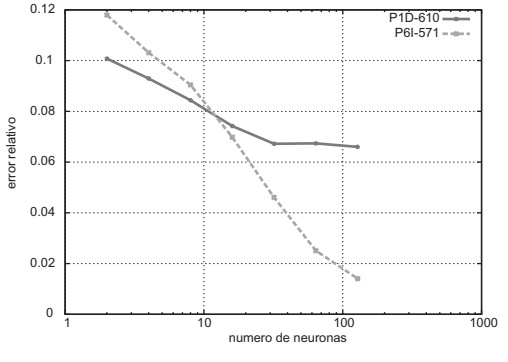

Figura 6. Variación del error relativo al variar el número de neuronas de la capa oculta del modelo general.

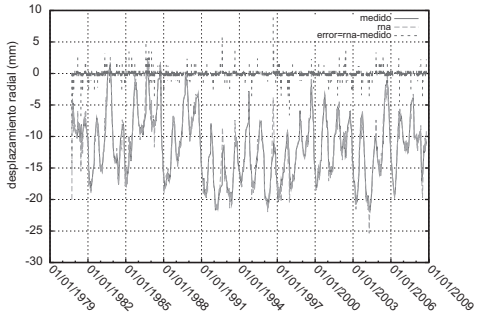

Figura 7. Desplazamiento radial del péndulo 1D a la cota 610 msnm simulado con el modelo general de 128 neuronas en la capa oculta.

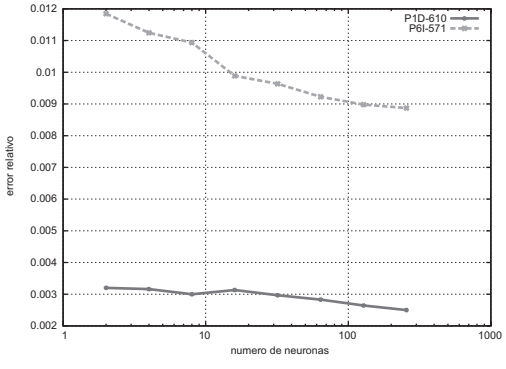

Figura 8. Variación del error relativo al variar el número de neuronas de la capa oculta del modelo de corto plazo.

Se observa que el error en este modelo es siempre menor que el del modelo estadístico.

En la figura 7 se muestra la aproximación realizada mediante una red neuronal de 128 neuronas en la capa oculta del modelo general para aproximar el desplaza-miento radial del péndulo 1D en la cota 610 msnm.

5.5 *Modelo de corto plazo*

Es el modelo más complejo de los estudiados. Consta en la capa de entrada de todas las variables del modelo general más los desplazamientos de los tres días anteriores al que se predice. Es un modelo de actualización diaria, por lo tanto es el más costoso computacionalmente, ya que se entrena la red diariamente para predecir los desplazamientos del día siguiente. Incluir estos tres desplazamientos es equivalente a dar información respecto a la tendencia del movimiento del péndulo. Al igual que en el modelo general, con esta configuración se permite a la red ponderar la importancia de las distintas

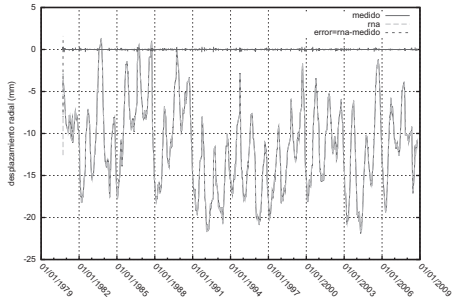

Figura 9. Desplazamiento radial del péndulo 1D a la cota 610 msnm simulado con un modelo de corto plazo.

medias móviles en el com-portamiento de la presa sin que haya un preproceso fuera del ámbito de la red. Usando este modelo se obtienen resultados con errores varios ordenes de magnitud menores que con los demás modelos. Esto se debe a que se predicen los desplazamientos a muy corto plazo.

La figura 9 muestra los resultados obtenidos al usar la arquitectura del modelo de corto plazo y una capa oculta de 256 neuronas para el péndulo 1D a la cota 610 msnm.

6 CONCLUSIONES

Se ha representado el comportamiento de la presa mediante los desplazamientos de sus péndulos, y se han obtenido modelos estadísticos de estos. Se ha definido un modelo simple, uno general y uno de corto plazo para los desplazamientos de los péndulos usando redes neuronales. Se ha comprobado que con los modelos de redes neuronales se obtienen resultados que pueden ser iguales o mejores que los del modelo estadístico. Se ha constatado que el modelo general y el de corto plazo tienen un mayor coste computacional que el modelo estadístico. El modelo general tiene una mayor flexibilidad en su aplicación que el modelo simple y el estadístico.

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Predicting concrete arch dams stresses in linear domain using Endurance Time method

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ABSTRACT: At present study, Endurance Time (ET) method that is a new dynamic pushover procedure is introduced and its application in predicting concrete arch dam behavior in linear domain is investigated. In this method structure is subjected to gradually intensifying acceleration functions. DEZ arch dam was selected as case study. FE model of dam-reservoir-foundation was excited in three levels of MCL, MDL, DBL using six records scaled based on design spectrum of the site and results were compared with those obtained from ET analyses. It was found that results obtained from ET method at equivalent target time are in good agreement with extreme results obtained from exciting the system using natural records. It can be concluded that using ET method reduces time of analyses and there is acceptable correspondence between ET and time-history analyses results.

1 INTRODUCTION

In recent years, developing innovative methods for analyzing structures which are able to predict responses in minimum time with high accuracy in different hazard levels have been interested by structural engineers. One of these methods is Performance Based Seismic Engineering PBSE. Various procedures were introduced for this purpose. Nonlinear static analysis or pushover analysis is one of these methods in which seismic demand is computed by nonlinear static analysis of structure subjected to monotonically increasing lateral forces until a target value of specific point displacement is reached (Chopra et al., 2003). Incremental Dynamic Analysis IDA is another approach in which the seismic load is scaled and different nonlinear dynamic analyses are performed to estimate dynamic performance of global structural system (Vamvatsikos & Cornell, 2002). Using this method Damage Measures (DMs) of structures are obtained at different Intensity Measures (IMs) and performance of structures can be reviewed more precisely. The main problem of the method is the required large number of nonlinear dynamic analyses to obtain reliable results.

Endurance Time method is basically a simple dynamic pushover test that tries to predict DMs of structures at various IMs by subjecting them to some predesigned intensifying dynamic excitations. The excitation in ET method is named as acceleration function instead of accelerogram to prevent confusion with real ground motions and simulated accelerograms which are usually compatible with real ground motions. ET acceleration functions are designed in a manner that their intensity increases through time (Riahi & Estekanchi, in press). Because of increasing demand of ET acceleration function, structure (or system) gradually goes from linear elastic to nonlinear phase, finally leading to global dynamic instability (Riahi et al., in press). If ET acceleration functions can assemble the major characteristics of real ground motions with different intensities, they can provide a good estimate of the real response of structures to be used in PBSE.

Application of ET method in linear seismic analysis of structures has been studied before (Estekanchi et al., 2007, Application …). Validity and accuracy of this method in nonlinear seismic analysis of SDOF structures was investigated as well (Riahi et al., in press). It must be mentioned that all accessible papers are limited to simple steel or concrete frames and their damage indexes are limited to those pointed in relevant codes. Madihi was studied application of ET method in design and analysis of concrete gravity dams (Madihi, 2008). He found that responses obtained from ET and TH (Time History) analyses are close in linear range. However, definitions of criteria that can help to predict responses in nonlinear range depend on the geometry of the dam. Amirpour was studied

performance of arch dams in linear domain using ET method (Amirpour, 2009). He compared results of ET with TH analyses using parameters like DCR and cumulative inelastic duration for the purpose of performance evaluation of AMIR-KABIR dam. Zarei considered effect of vertical component of earthquake in nonlinear behavior of gravity dams using ET method (Zarei, 2009). Hariri Ardebili studied nonlinear seismic performance of arch dams using ET method (Hariri Ardebili, 2010). He modeled DEZ arch dam taking into account material and joint nonlinearity and conducted several ET and TH analyses relevant to various hazard levels of the dam site.

In this paper ability of ET method in predicting high concrete arch dam principal stresses in linear domain (excluding material and joint nonlinearity) is considered. Concept of ET criteria is first explained and prospective methods to implement them are discussed. Parallel with ET analysis, Time-History (TH) analysis is performed to evaluate results obtained from two methods. For this purpose 203 m DEZ high concrete arch dam was selected as case study. Fluid-Structure Interaction (FSI) effect of reservoir water is considered by modeling water as compressible material. Moreover for considering flexibility of foundation, rock is simulated as mass-less medium. FE model of the considered system was exited in three operational levels as MCL, MDL, DBL using six records which are scaled based on design spectrum of the site and their results are reported as time-history analysis. On the other hand the system was exited by three acceleration functions in endurance time analyses.

2 CONCEPT OF ENDURANCE TIME METHOD

Endurance Time method is a simple dynamic pushover procedure to predict seismic performance of structures by analyzing their resilience when subjected to predesigned intensifying dynamic excitations. In this method, numerical or experimental models of structures (or systems) are subjected to intensifying dynamic accelerations. Structural responses, such as displacements, velocities, accelerations, stresses or other appropriate damage measures are monitored up to the point where the structure collapses or analysis does not converge. The time duration from the start of test or analysis to this collapse point is called endurance time. Basically, longer the structure can endure imposed excitations, it is judged to have better performance. In practice, the analysis or experiment need not be continued until the real collapse of the structure. Any convenient performance parameter, such as maximum displacement, stress ratio and so on can be considered and analysis or experiment can be commenced until the desired level of excitation has been covered (Estekanchi et al., 2004, Endurance …; Estekanchi et al., in press, Endurance …).

Concept of ET analysis can be described using hypothetical shaking table experiment on simple frames. Three different structures with unknown structural properties are to be ranked according to their seismic resistance performance. All three structures are fixed on a shaking table and the test begins by subjecting the structures to an acceleration function, as shown in Figure 1. According to lapse of time, the amplitude of vibrations increases in the shaking table.

As the vibration amplitude increases, in $t = t_1$ one of the structures fails as shown in figure. As the amplitude of vibration is constantly increased, in $t = t_2$ and $t = t_3$ the second and third structures will also fail. Based on these results, and considering that the lateral loads induced by the shaking table somehow correspond with earthquake loads, structure 'A', which failed earliest, is ranked as the

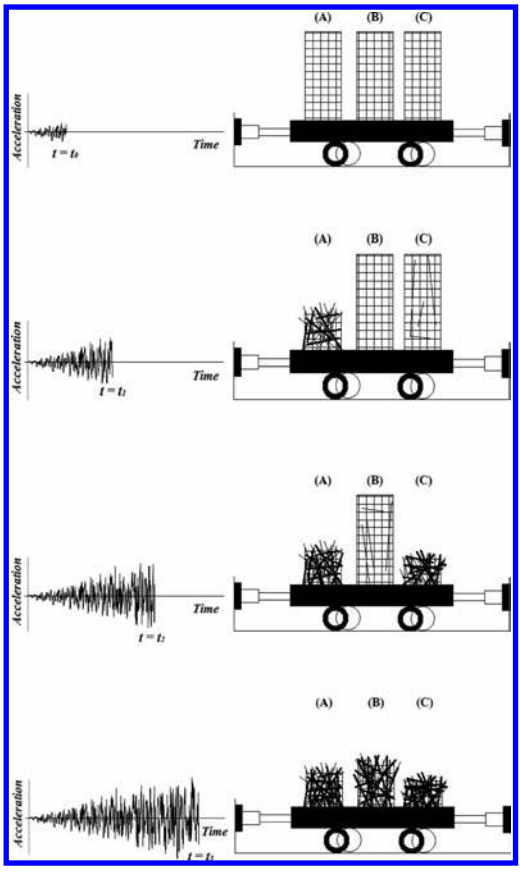

Figure 1. Schematic of hypothetical shaking table test.

worst and structure 'B', which endured longest, is ranked as the best one. This hypothetical experiment describes the essence of Endurance Time method.

Numerically, after simulation an appropriate finite element model of physical phenomenon and specification of suitable damage index, ET analysis can be run using an appropriate ET accelerations functions. Time-history of the selected damage index is recorded during analysis. Maximum absolute value of endurance index (or any arbitrary response of the system) is plotted for total duration of analysis. Then based on this information Endurance Time Curve (ETC) is plotted for each damage index or response of the system. In theory, ETC represents a special diagram that its vertical axe's values refer to the maximum absolute values of damage index or a particular response of system during the time interval from 0.0 to t. The hypothetical ETC for mentioned frames that can obtain from numerical analysis are shown in Figure 2. As can be seen in this figure for the same damage index (for example maximum drift of roof, base shear and so on) frame B has longest endurance time (about 17.5 s) and frame A has shortest endurance time (about 13.0 s).

One of the most important issues in successful implementation of the procedure is determination of a suitable acceleration function so that the results from ET analysis (or testing) can be correlated reliably well with the response of the structure subjected to real earthquakes. For this purpose, concept of response spectra is taken advantage of in ET method (Estekanchi et al., 2007, Application …; Estekanchi et al., 2007, Estimating …). Optimization techniques are used in order to create ET acceleration functions with the property of having a response spectra which proportionally intensifies with time while remaining compatible with a pre-specified template response spectra curve. This means that the response spectrum of any window of the acceleration function from $t_0 = 0$ to $t_1 = t$ resembles that of the target spectrum with a scale factor which is proportional to time t (Estekanchi et al., 2007, Application …). ET acceleration functions which is used in this research are designed in such a way that their response spectrum remains proportional to that of the average of seven real earthquakes recorded on a stiff soil condition (Riahi et al., in press).

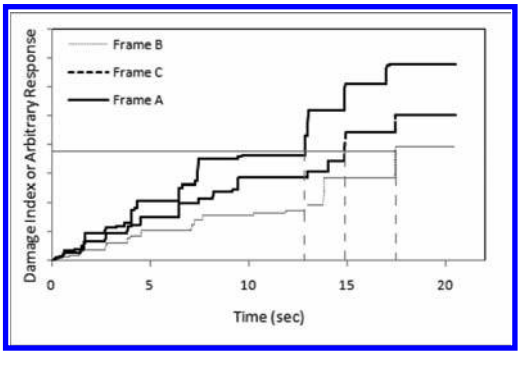

Figure 2. Hypothetical ETC for frames in Figure 1.

3 SIMULATION OF FINITE ELEMENT MODEL

DEZ is 203 m high double curvature arch dam with a peripheral joint separating the dam body from a concrete saddle structure. The dam is located in a narrow gorge at DEZ River, in Khuzestan Province in Iran, about 150 km upstream of provincial capital of Ahwaz. Crest length is 240 m and thickness at the crest level is 4.5 m.

The finite element idealization prepared for dam and foundation rock is presented in Figure 3. The model consists of 792 solid elements for modeling concrete dam and saddle and 3770 solid elements for simulation of foundation. The 8-node solid elements have three transitional degrees of freedom at each node. In addition, the reservoir is modeled using 3660 fluid elements. Utilized Eulerian fluid elements have three translation DOFs and one pressure DOF in each node. For exerting suitable boundary condition to system all nodes in far boundary of the foundation are restricted in three transitional directions.

The isotropic elasticity for mass concrete in static and dynamic condition is 40 GPa and 46 GPa respectively. Poisson's ratios are 0.2 and 0.14 and the density of concrete is taken as 2400 Kg/m^3. Isotropic elasticity of foundation rock material in saturated and dry conditions is 13 GPa and 15 GPa, respectively. Reservoir water density is assumed 1000 kg/m^3, sound velocity is 1440 m/s in water and wave reflection coefficient for reservoir around boundary is supposed 0.8, conservatively.

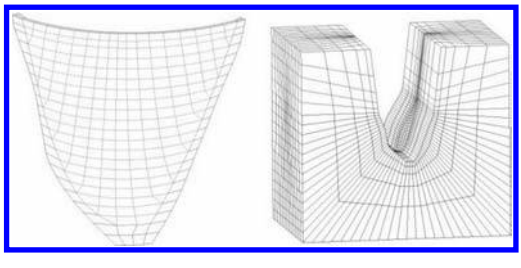

Figure 3. Finite element model of dam and foundation.

4 SPECIFICATION OF GROUND MOTIONS AND ACCELERATION FUNCTIONS

To investigate potential of ET method in estimating response of real ground motions, a set of ET acceleration functions are used which were generated using average response spectrum of real ground motions. To reach this goal, 20 accelerograms which were recorded on site class C by the NEHRP, and used in FEMA 440 are selected (FEMA-440, 2005). From these ground motions, 7 records that their response spectra shapes were more compatible with the response spectrum of soil type II of INBC standard 2800 are selected (Riahi & Estekanchi, 2006). These 7 accelerograms were scaled to produce response spectrum that is compatible with INBC standard 2800 spectrum. Finally average of pseudo acceleration spectrum of these scaled accelerograms was obtained and smoothed. The smoothed spectrum was used as the target spectrum in generating new ET acceleration functions. It is important to note that the ET response spectra remains proportional to the target spectra obtained from seven real earthquakes.

All FE models are analyzed in 3D space but subjected to a single horizontal component of ground motion (Upstream/Downstream direction). In TH analyses, the system was excited using six real earthquakes in three hazard levels of DBL, MDL, and MCL (for each level two records). On the other hand, the system was excited using three acceleration functions in ET analyses. It must mention that for each natural earthquake record and acceleration function analyses were conducted in two opposite directions.

5 SUITABLE EQUIVALENT TARGET TIME FOR HAZARD LEVELS

Key factor in prosperity of ET method lay on determination of suitable equivalent target time for various hazard levels. The response of arch dam in each window of time is corresponding with a certain hazard level. Ordinary low target times are corresponding with low hazard levels and vice versa. Generally there are two approaches in calculation of target time, i.e. methods based on concept of spectrum analysis and statistics methods.

Because of the fact that ET fundamental concepts come back to spectrum analysis, it is possible to use response spectrum of MDOF systems in determining equivalent target time $t_{eq}(T_R)$ for considered hazard levels in which T_R is return period of earthquake and refers to dam three main hazard levels. In this approach, equivalent target time is determined using mode superposition and considering damage index in linear domain and CQC method is used for combination of correlated modes of dam-reservoir-foundation system.

Table 1. Equivalent target time for hazard levels.

Hazard level	Suggested target time
DBL	$t_{eqIII}(DBL) = 3.25$ s
MDL	$t_{eqIII}(MDL) = 4.17$ s
MCL	$t_{eqIII}(MCL) = 6.36$ s

The other way for calculation of appropriate target time is usage of statistics methods. The first step is selection of suitable range of periods in order to comparison of ET and desired hazard levels response spectra. In many instructions and codes the period range that includes more than 90% of total mass of system is considered as main range. In second step target time is chosen so that the surface enclosed between the two ET and the desired hazard level response spectra including the percentage of the effective mass at each period in the selected range is set to zero (or minimum possible value). With this method the equivalent target time for hazard levels can be written as Table 1. This approach is based on arithmetic mean weight concept and is shown as $t_{eqIII}(T_R)$.

For example, based on Table 1 the average of maximum absolute values for stresses in TH analysis of dam-reservoir-foundation using MANJIL and TABAS earthquakes (Natural earthquakes in MCL) should have the same or close values to those obtained from average of three acceleration functions in ET analyses until equivalent target time in MCL, i.e. $t_{eqIII}(MCL) = 6.36$ s. On the other words ET analyses are capable to predict TH analyses results in MCL until target time equal to 6.36 s. This rule is indefeasible in other hazard levels.

6 PREDICTING TIME-HISTORY ANALYSIS RESULTS WITH ENDURANCE TIME

Figure 4 represents time-history of principal stresses at crest center for real earthquake records in DBL. It is noteworthy that for each earthquake two different analyses were conducted; the first in +US/DS direction and the second in –US/DS. Figures 5 and 6 show the same plots in MDL and MCL. Table 2 represents extreme values for maximum and minimum principal stresses extracted from these figures.

Figure 7 shows double Endurance Time Curves (ETCs) for maximum and minimum principal stresses at crest center from endurance time analyses. The rising graph shows maximum principal

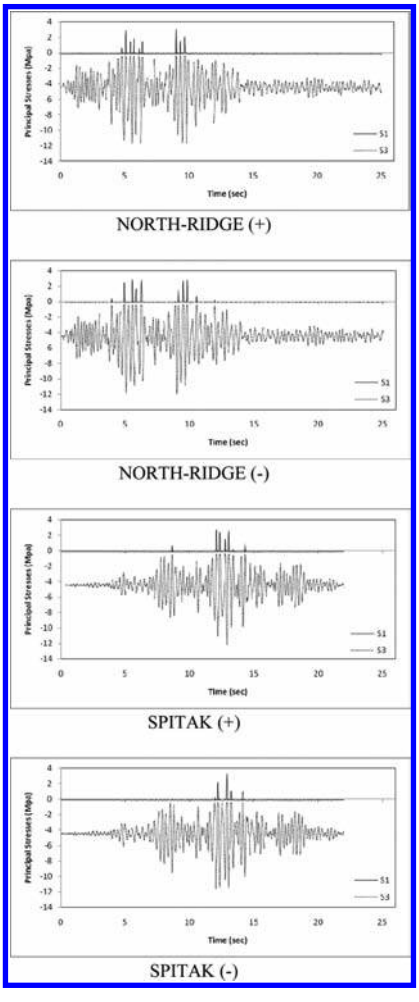

Figure 4. Time-history of principal stresses at crest center; DBL.

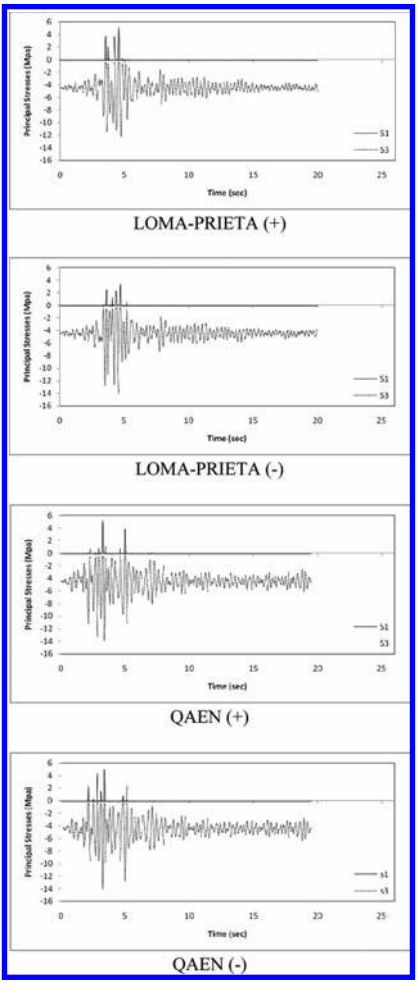

Figure 5. Time-history of principal stresses at crest center; MDL.

stress and the falling one shows minimum principal stress, respectively. Also linear trend-line is shown for each ramp-shaped graph. This linear trend-line moderates some extreme values in corner of ramps.

Table 3 represents maximum principal stresses from ETCs until equivalent target time for various hazard levels according to real ETCs and linear trend-lines. Table 4 represents percentage of errors from each acceleration function and average of real earthquake records, respectively. As can be seen the values of stresses from pair records in hazard levels have differences with each other. These differences are meaningful for MCL in comparison with MDL and DBL.

Comparing results and percentage of errors from Tables 3 and 4 it is found that using linear

Table 2. Extreme values for maximum and minimum principal stresses extracted from real earthquakes.

		Principal stresses (MPa)	
		Maximum	Minimum
MCL	MANJIL	14.42	–23.31
	TABAS	9.18	–18.07
	AVE	11.80	–20.69
MDL	QAEN	5.17	–14.06
	LOMA-PRIETA	5.10	–14.00
	AVE	5.13	–14.03
DBL	NORTH-RIDGE	3.03	–11.92
	SPITAK	3.28	–12.17
	AVE	3.16	–12.04

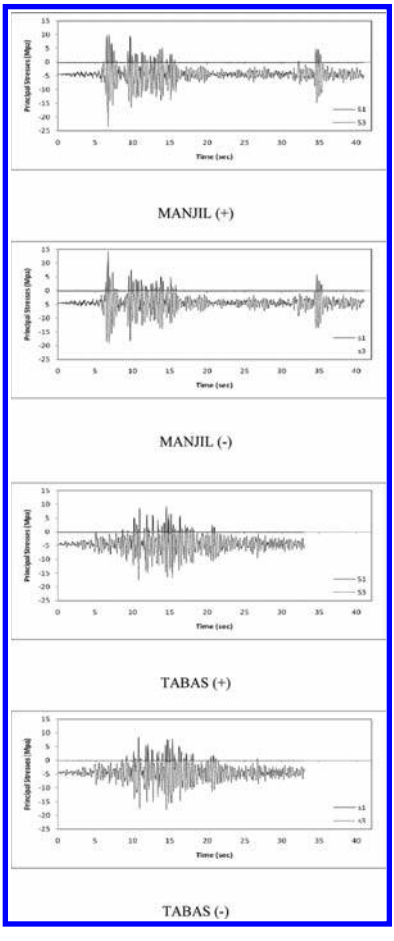

Figure 6. Time-history of principal stresses at crest center; MCL.

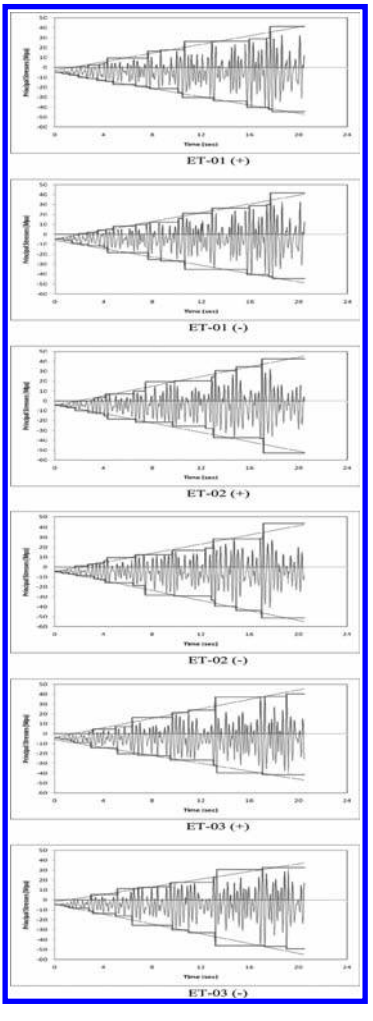

Figure 7. Double Endurance Time Curves for maximum and minimum principal stresses in center of crest.

Table 3. Maximum principal stresses from ETCs until equivalent target time for various hazard levels.

ET NO	The kind of ET curve	Maximum principal stress at equivalent target time		
		DBL	MDL	MCL
ET-01	Real	2.41	4.48	9.57
	Linear	4.13	6.15	10.93
ET-02	Real	3.18	6.34	9.54
	Linear	3.88	4.95	10.09
ET-03	Real	4.70	5.51	11.66
	Linear	4.28	6.04	11.80
ET-AVE	Real	3.43	5.44	10.26
	Linear	4.10	5.71	10.94

trend-line reduce the errors in MCL but increase errors in MDL and DBL. Percentage of errors from real ETC in ET-AVE for DBL and MDL are 8.6% and 6.0%, respectively. Also errors in MCL from ET-AVE for real ETC and linear trend-line are –13.1% and –7.3%. Comparing the sign of errors in three levels shows that generally the values of maximum principal stresses from TH analyses are more than ET method in MCL. In addition maximum principal stress has greater value in MDL and DBL from endurance time analyses.

Table 5 represents minimum principal stresses from ETCs in equivalent target time for various hazard levels according to real ETCs and linear trend-lines. Table 6 represents percentage of errors from each acceleration function and average of real earthquake records, respectively.

Table 4. Percentage of errors in each acceleration function and average of real earthquake records in maximum principal stress.

ET NO	The kind of ET curve	Percentage of errors in ET to real earthquakes average DBL	MDL	MCL
ET-01	Real	–23.7%	–12.8%	–18.9%
	Linear	30.7%	19.8%	–7.4%
ET-02	Real	0.6%	23.5%	–19.2%
	Linear	22.8%	–3.6%	–14.5%
ET-03	Real	48.8%	7.3%	–1.2%
	Linear	35.5%	17.6%	0.0%
ET-AVE	Real	8.6%	6.0%	–13.1%
	Linear	29.7%	11.3%	–7.3%

Table 5. Minimum principal stresses from ETCs until equivalent target time for various hazard levels.

ET NO	The kind of ET curve	Minimum principal stress at equivalent target time DBL	MDL	MCL
ET-01	Real	–11.28	–13.36	–18.46
	Linear	–11.93	–14.27	–19.86
ET-02	Real	–12.06	–15.24	–18.43
	Linear	–11.05	–13.23	–18.45
ET-03	Real	–14.41	–14.41	–22.38
	Linear	–12.42	–14.26	–20.12
ET-AVE	Real	–12.58	–14.34	–19.76
	Linear	–11.80	–13.92	–19.48

Table 6. Percentage of errors in each acceleration function and average of real earthquake records in minimum principal stress.

ET NO	The kind of ET curve	Percentage of errors in ET to real earthquakes average DBL	MDL	MCL
ET-01	Real	–6.4%	–4.8%	–10.8%
	Linear	–1.0%	1.7%	–4.0%
ET-02	Real	0.1%	8.6%	–10.9%
	Linear	–8.3%	–5.7%	–10.8%
ET-03	Real	19.6%	2.7%	8.1%
	Linear	3.1%	1.6%	–2.8%
ET-AVE	Real	4.5%	2.2%	–4.5%
	Linear	–2.0%	–0.8%	–5.9%

As it is evident from Tables 5 and 6, the percentage of errors in compressive stresses are less than errors in tensile stresses. Moreover, in MCL ET method gives lower values than TH analysis but in MDL and DBL this is vice-versa. Generally using linear trend-line reduces the errors in comparison with real ETC. Errors from real ETC in ET-AVE for DBL, MDL and MCL are 4.5%, 2.2% and –4.5% respectively.

Figure 8 shows the percentage of errors along the upstream nodes of crown cantilever for maximum and minimum principal stresses. As can be seen in this figure the variations of errors are severe for tensile stresses than compressive stresses. Percentages of errors are limit to 40% in all cases. Also unlike the expectations, the errors are less in MCL than MDL and DBL.

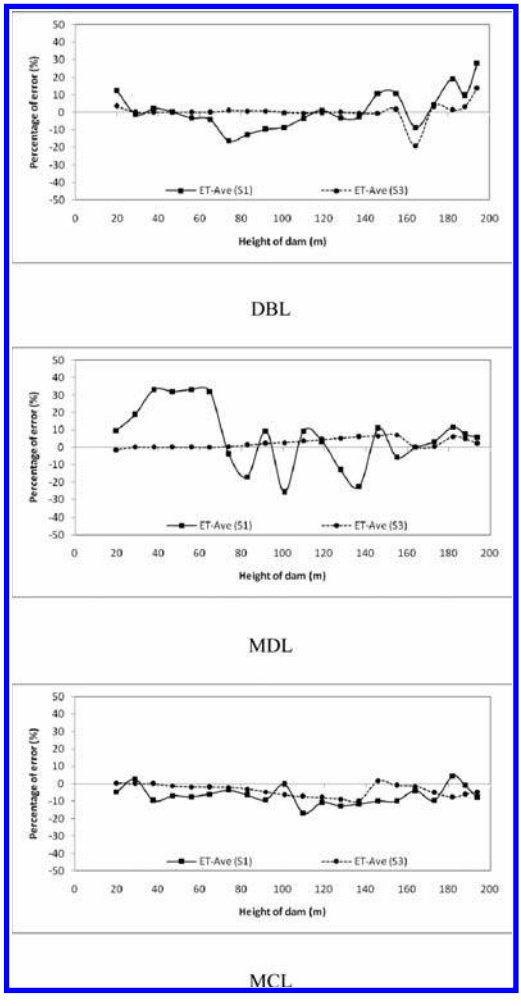

Figure 8. Percentage of errors variation along the crown cantilever for maximum and minimum principal stresses.

The main advantage of endurance time method is decreasing both number and time of analyses than time-history analysis. The total time of six real earthquake records is 160.5 s, considering time step equal to 0.02 s lead to have about 8000 load steps. On the other hand according to Table 1 maximum target time is 6.36 s. Conducting ET analyses with first 8.00 s of ET acceleration functions and considering time step equal to 0.01 s lead to have about 2400 load step in ET analyses which is 30% of those needed in time history analysis. On the other hand conducting six analyses with three acceleration functions as ET method can reduce 50% in number of analyses than to TH method (conducting twelve analyses with six real earthquake records).

7 CONCLUSION

At the present study, ability of endurance time method in predicting concrete arch dam maximum and minimum principal stresses at a selected location in linear domain was investigated. DEZ dam in Iran was used as case study and the dam body, foundation medium and the reservoir is modeled using appropriate finite elements. Based on the conducted dynamic analyses results, it is found that the stresses of the system extracted from ET method up to the equivalent target time are in relatively good agreement with the extreme principal stresses extracted from time-history analyses of the system. The errors for central cantilever are in reasonable range and the values for compressive stresses in ET method have more correspondence to time-history analysis results than tensile stresses. Although, the variation of errors is the same for various hazard levels in compressive stress, generally, the errors of tensile stress along the cantilever for MCL are less than MDL and DBL. Furthermore using linear trend-line increases percentage of errors for low hazard levels and vice versa. Finally, the great advantages of endurance time method lays on low cost of analysis than a set of time-history analyses especially when the system should be analyzed in various hazard levels.

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Value of diligent visual inspections for the evaluation of dam behaviour

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ABSTRACT: The evaluation of the behaviour of dams comprises both quantitative as well as qualitative aspects. For a number of dams, diligent visual inspections are the only practical method of monitoring and evaluating their behaviour. The Department of Water Affairs of South Africa developed and implemented a multi-level visual inspection programme (including a logbook system to capture observations during these visual inspections) 25 years ago. The effectiveness of the system has since degraded due to a number of reasons. Some of the possible causes are discussed briefly. The impact of these issues; the importance of regular review of information; the proper training and motivation of staff and sensitising of decision makers are addressed before finally concluding with observations made and lessons learnt in this regard.

1 VALUE OF VISUAL INSPECTIONS

The evaluation of dam behaviour includes both quantitative as well as well as qualitative aspects and relies heavily on continuous monitoring during the lifetime of the structure and the regular relevant interpretation of the monitoring results.

For the majority of the 340 large dams owned by the Department of Water Affairs of South Africa (DWA), diligent visual inspections and evaluation of their behaviour during their lifetime are the only practical method of monitoring.

The importance of institutional memory in the monitoring and evaluation of the behaviour of dams should not be under-estimated. Visual inspections are one of the important means to develop institutional memory. Goguel (2007) provided a graphical presentation of the development of institutional memory as well as the difference between theoretical institutional memory and living memory. This graph has been extended to show the actual institutional memory decay with time (with and without proper visual inspections, see Figure 1).

Visual inspections, especially at dams where it is the only method of continuous monitoring forms the most important form of creating institutional memory during the operational life span of a dam. Non-performance of visual inspections creates a situation where no additional or very little theoretical institutional memory is added during that period in the lifespan of the dam.

This in turn creates the situation where the living memory is also significantly less than when diligent visual inspections would have taken place on a regular basis. The loss of theoretical institutional memory due to the lack of monitoring (visual and observation of instruments) as well as the loss of the "living" memory of retired or diseased staff members is permanent. It is lost forever and cannot be generated again.

2 VISUAL INSPECTION PROGRAMMES

Like many other countries, the Department of Water Affairs of South Africa developed and implemented a multi-level visual inspection programme 25 years ago. The programme is based on a comprehensive logbook system not only to capture observations during these visual inspections, but also a track record of the reporting of problems and by whom and when it was attended to. It is important to note that the success of such a programme, to a large extent, relies on having a sufficient number of dedicated personnel with relevant experience and sufficiently qualified for the task (but not over-qualified).

This inspection programme comprises five levels of "inspections" (a graphical representation is given in Figure 2):

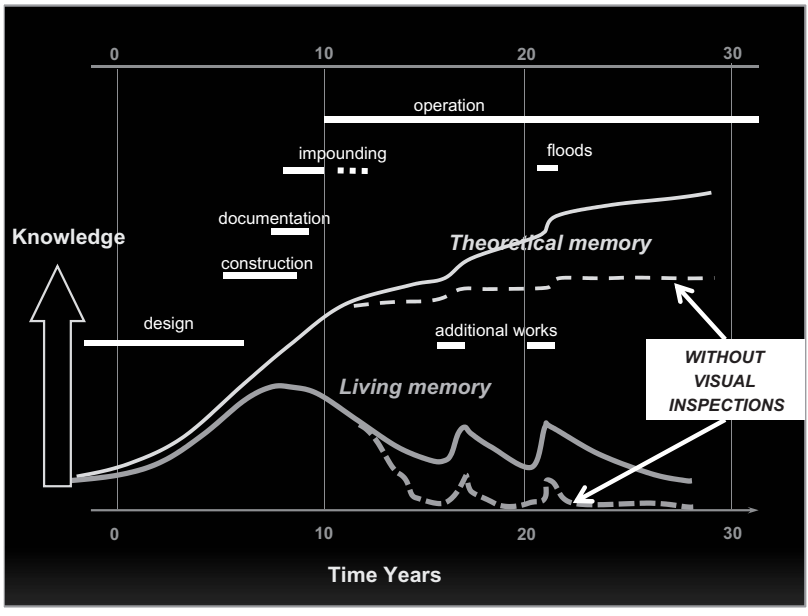

Figure 1. The creation of theoretical institutional memory and living memory during the life span of a dam (Goguel: 2007) including the influence of not carrying out regular visual inspections.

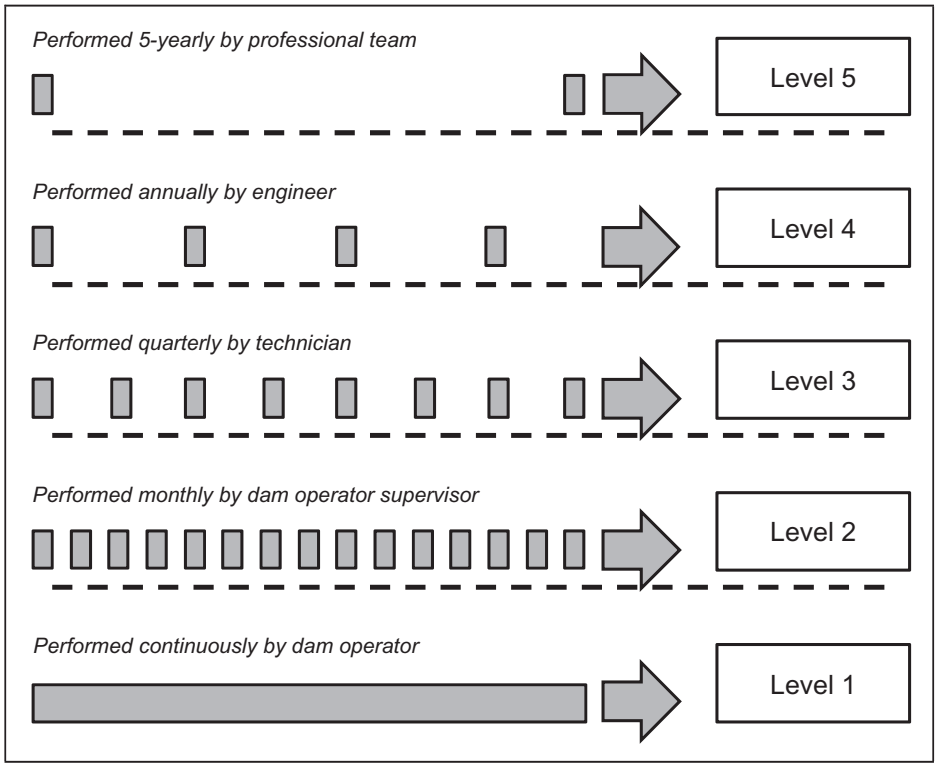

Figure 2. Multi-level visual inspection programme.

- Level 1 visual inspections of the whole dam are performed by the dam operators on a continual basis and includes the formal capturing of these observations in the logbook preferably on a weekly basis;
- Level 2 visual inspections are done by the dam operator's supervisor on a monthly basis by checking and validating the observations of the previous level (preferably a person that gained sufficient experience working as a dam operator);
- Level 3 visual inspections are done by a technician on a quarterly basis validating the observations of the two previous levels. In addition to capturing the observations in the logbook a single page inspection report highlighting any problems at the dam are compiled by the technician and forwarded to the engineer (next level); and
- Level 4 visual inspections are done on an annual basis by the engineer responsible for the dam. In addition to capturing the observations in the logbook a single page inspection report highlighting any problems at the dam are compiled by the engineer and forwarded to the next level; and
- Level 5 visual inspections are performed by a team of professionals (lead by an approved professional person) on a five yearly basis and forms part of the five yearly dam safety evaluation required by dam safety legislation (RSA 1998).

Each higher level of inspection should include the persons responsible for the lower levels of inspection. For example during the 5-yearly inspections the dam operator, dam operator supervisor, technician and engineer should accompany the professional team. These higher levels of inspections incorporate and involve everyone that forms part of the lower levels of inspection. They would share their institutional memory and on the other hand they may receive some education/training or "calibration" of their judgement.

These logbooks also provide valuable history of problems experienced (in chronological order as well as the attention they received and by whom). Problems recorded in the logbooks have to be signed off by higher levels of authority and finally by the person in charge of those who rectified them. The appropriate procedures are included in the operational and maintenance manuals for each dam (as discussed below).

The main philosophy of the logbook is to capture changes in the behaviour or the condition of the dam compared to previous visual inspections. Each dam has a site-specific logbook comprised of the following:

- The main part is in a table format with the different components described in the vertical headings and provision is made for the inspection dates in the vertical headings.
- The inspection of a specific component is indicated by either a tick (inspected and in order) or a slash (if not inspected) or a cross (when a problem or change was detected). The latter is then reported on on the comments page. These changes are described in more detail and if immediate attention is not required the higher level of authority signs off during the particular level of inspection. The tables in the logbook are used for levels of inspection;
- Provision is also made in a separate table for visits/inspections by technical staff and their observations.

These logbooks also serves the purpose of a legal document that is living proof of whether the operator/supervisor/technician/engineer has done what would have been expected of the specific "reasonable person".

In addition, concise "easy to comprehend" and to the point operation and maintenance manuals have been compiled using ample photographs to avoid confusion. In each of these site-specific manuals, details are provided on the different components to be inspected and the frequency of these visual inspections. These manuals proved to be a valuable source of information for dam operators with regards to visual inspection requirements as well.

3 FACTORS IMPACTING NEGATIVELY ON THE EFFECTIVENESS OF VISUAL INSPECTIONS

Initially these inspection programmes functioned successfully during the first few years of their existence. Then, because of a number of reasons, the effectiveness initially stagnated and thereafter started to deteriorate slowly and later exponentially.

Some of the general reasons for the deterioration can be ascribed to: the generation gap between the retiring staff members and their successors; relatively low remuneration as a result of inappropriate job evaluations resulting in low morale; attitude and work ethics (of especially younger staff members) and finally a high turnover of staff.

Inappropriate levels of education and experience requirements together with the lack of refresher courses are considered as the main compelling reasons behind the deterioration phase. Over-qualified/ under-experienced as well as under-qualified/ under-experienced staff, tends to neglect visual inspections. In the first instance high levels of frustration and high turnover in these staff members are experienced.

For dam operators and their immediate supervisors' diligence, dam safety experience (especially of the particular dam) and a positive attitude

are of essence and definitely not a high level of education. Supervisors should ideally have progressed through the ranks, from dam operator onwards. For those in posts responsible for level 3 and 4 inspections (technicians and engineers) relevant qualifications is required but several diligent and intelligent individuals have progressed by means of appropriate experience and self-education to these levels. Due diligence is however of utmost importance.

Another reason for experiencing low morale and due diligence can be ascribed to the lack of recognition for the value of their routine visual inspections. Experience has shown the effectiveness of applying the *Hawthorne effect* (Landsberger 1958) with regards to monitoring and visual inspections in particular. In other words to give attention to them and the importance of their input during the regular review of information, as well as regular feedback, proper training through refresher courses for motivational purposes. (The *Hawthorne effect* is a form of reactivity whereby subjects' show an increase in productivity due to the motivational effect of interest shown in them).

Observations made by the authors when conducting 5-yearly dam safety inspections (especially during the last ten years) show a direct relationship between the quality of the visual inspection programme and the general condition of the dam. In general where the visual inspection programme is neglected, the general condition of the dam is, as a rule, also in a state of neglect.

4 CONCLUDING REMARKS

Experience has highlighted the importance of visual inspections for the development and accumulation of a wealth of institutional memory. The important role it plays in the evaluation of dam behaviour especially where visual inspections are the only form of monitoring cannot be overemphasised.

The implementation of an effective and successful visual inspection programme depends not only on a properly designed formal process, but more so in the appointment of appropriate personnel (diligent with the appropriate levels of experience and education for each of the levels of responsibility). The success in the long-term can be achieved by applying the Hawthorne effect (Landsberger 1958). These staff members must receive praise when it is due. Even reprimanding can be done in a positive manner. These members must receive full recognition and be directly involved during regular reviews of information. Regular feedback on the behaviour of the dam as well as on their performance, proper training by means of refresher courses and finally regular motivation of all staff responsible for visual inspections are all essential.

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Procedimientos fotogramétricos para inspecciones de paramentos de presas. Aplicación a la presa de San Esteban. Campañas 2001y 2006

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ABSTRACT: The Crakview is an indispensable tool in the management of maintenance of large concrete structures; that allows documentation, quantification and evolution of defects in time, establishing a life management structure. It has born as a result of the application and improvement of techniques for mapping and photo interpretation to the inspection of these structures, together with the development of powerful computers, a cost reduction in software processing and high performance image acquired by digital photography.

1 INTRODUCCIÓN

La fotogrametría tuvo sus inicios en el levantamiento de fachadas arquitectónicas y plantas de edificios, mediante el uso de fotografías terrestres, incorporando seguidamente las fotografías aéreas para el levantamiento de la cartografía base, siendo la mayor parte de la cartografía topográfica de nuestro planeta realizada mediante esta técnica.

Como consecuencia de la utilización de la fotografía aérea, se desprendió de la fotogrametría la disciplina de la fotointerpretación, la cual comparte sus fundamentos básicos con la fotogrametría aérea.

A partir de los años ochenta, el desarrollo acelerado de la informática, condujo al establecimiento de la teledetección como consecuencia lógica de la evolución de la fotointerpretación, así como al desarrollo de técnicas de tratamiento informatizado de imágenes digitales y al desarrollo de la visión por ordenador.

Actualmente, con el apoyo de la informática y la reducción de los costos de programas de tratamiento de imágenes, la fotogrametría se ha convertido en una disciplina indispensable en campos como la inspección de grandes estructuras.

1.1 *Definición de fotogrametría*

La fotogrametría es la ciencia de realizar mediciones e interpretaciones fiables por medio de las fotografías, para de esa manera obtener características métricas y geométricas (dimensión, forma y posición), del objeto fotografiado.

La sociedad americana de fotogrametría y sensores remotos (ASPRS), define la fotogrametría como el arte, la ciencia y la tecnología de obtener información confiable de objetos físicos y su entorno, mediante el proceso de exponer, medir e interpretar tanto imágenes fotográficas como otras, obtenidas de diversos patrones de energía electromagnética y otros fenómenos.

1.2 *Inspección de grandes estructuras*

La evolución de la obra civil y en particular de la obra hidráulica, dificulta cada vez más la inspección de sus estructuras y paramentos verticales, debido a las cada vez mayores dimensiones alcanzadas. Esto conlleva el montaje y empleo de costosos medios auxiliares de elevación y personal muy especializado para la ejecución de los trabajos.

Si además, dicha inspección busca cuantificar la evolución del estado de la estructura, todos los medios deberán ser empleados de forma sistemática en periodos de tiempo establecidos (2, 3, 5, … años). Con ello, nos vemos forzados a repetir con esa misma periodicidad todo el proceso y determinar de una forma casi artesanal la evolución de fisuras, desconchones, eflorescencias, y corrosiones.

Derivado de todo lo expuesto, el mantenimiento de estas estructuras se convierte en algo que, aunque necesario, mueve importantes inversiones y complejos procedimientos.

Resulta evidente la importancia de un sistema que, por un lado, evite grandes inversiones

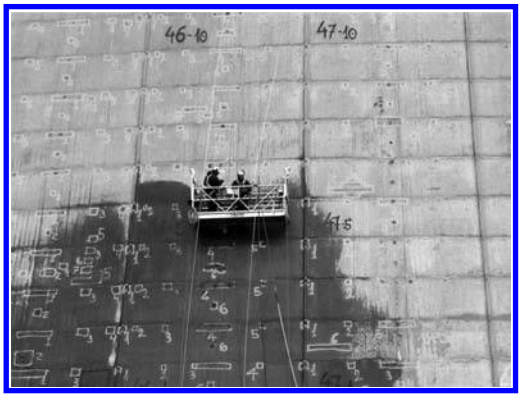


Figura 1. Inspección y marcado de defectos sobre paramento vertical mediante andamios colgados motorizados.

superfluas en elementos auxiliares y, por otro, facilite un seguimiento del paramento a estudio aprovechando datos de inspecciones previas. Como respuesta a esto, HCC adapta la técnica empleada en el mundo de la cartografía a la inspección detallada de estructuras de hormigón armado.

2 ANÁLISIS FOTOGRAMÉTRICO

Durante el año 2000, HCC desarrolla un sistema de análisis de grandes estructuras mediante videogrametría, que denominamos Crackview, permitiendo la observación y catalogación de cualquier fenómeno singular a distancias de 300 m.

Tras la filmación con cámara de video de alta definición y grandes ópticas del paramento, se procedía al fraccionamiento de la imagen y la obtención de miles de fotogramas para su estudio mediante programas de tratamiento de imágenes.

Con el desarrollo y alta calidad de resolución de la fotografía digital, en el año 2006 se realizo el cambio de filmación en video de alta resolución a barrido fotográfico digital de la superficie. El cambio trajo consigo una disminución sustancial del número de fotografías requeridas para el estudio, gracias a la gran resolución de las mismas.

El sistema mantiene las mismas fases de trabajo en ambos formatos.

2.1 *Modelización de la estructura*

La primera operación consiste en la realización de un modelo de alta precisión en 3 dimensiones de la estructura a estudiar. Partimos de la obtención de una serie de puntos de referencia que sean inalterables durante los siguientes años. Sus coordenadas se obtienen por procedimientos topográficos y/o fotogrametría terrestre. De este

Figura 2. Equipo de fotogrametría.

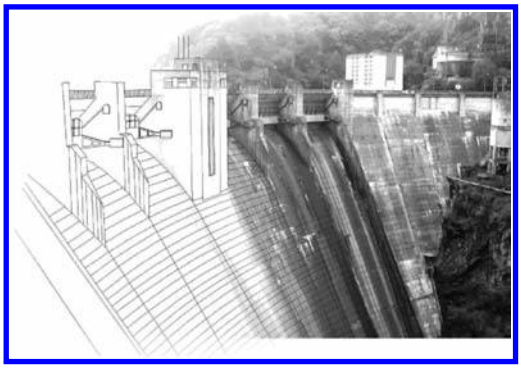


Figura 3. Modelización de la presa de San Esteban.

modo, conseguimos un patrón a escala sobre el que referenciar cualquier singularidad.

Este modelo es válido para el resto de inspecciones en años sucesivos, lo que abarata sensiblemente el coste total de la operación.

2.2 *Toma de datos*

Una vez obtenido el "lienzo" virtual sobre el que trabajar, se prosigue con el chequeo de la estructura mediante cámaras fotográficas digitales de alta resolución, dotadas de grandes ópticas. Con ello, nuestros técnicos disponen de los medios necesarios para que, usando como referencia puntos característicos del paramento, procedan a barrer la totalidad de la superficie, a distancias de hasta 300 m, llegando a apreciar fisuraciones de décimas de milímetro. En este proceso también se revela cualquier otro tipo de daño que pudiera existir (corrosiones, humedades, eflorescencias, vegetación, etc.).

Todas estas labores se llevan a cabo sin necesidad de medios auxiliares, andamios u otro sistema de elevación, con el ahorro económico y de tiempo que esto supone.

2.3 *Tratamiento de imágenes*

Tras obtener miles de fotografías de la estructura, la primera operación consiste en digitalizar dichos datos; ello proporciona una serie de ventajas directas, como pueden ser:

- Mayor vida del soporte digital frente a medios magnéticos. Los soportes tradicionales de foto ó vídeo poseen una vida limitada a unos pocos años, de gradándose gradualmente hasta la pérdida total de información.
- Duplicado de datos sencillo, sin pérdida de calidad. Los datos almacenados en medios analógicos convencionales van perdiendo calidad con cada copia; el medio digital copia y transmite íntegramente la información, sin limitación en el número de duplicados.
- Posibilidad de edición y mejora de los fotogramas. Todos los fotogramas obtenidos se funden en macroimágenes que combinan un gran tamaño con un importante grado de definición.

2.4 *Vectorización de las singularidades*

Ya en oficina, los técnicos se encargan de catalogar los defectos observados. Se procede a digitalizar las singularidades, asignándoles un código de colores para su calificación. Empleando los datos topográficos obtenidos en las primeras fases del trabajo "vectorizamos" dichos defectos, dotándolos de entidad tridimensional, coordenadas, longitudes y áreas.

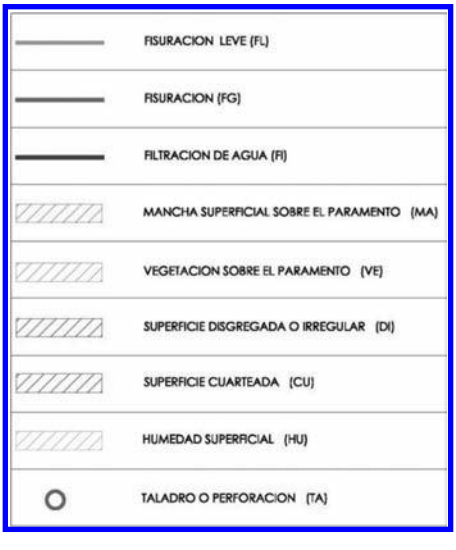


Figura 4. Catalogación de defectos.

2.5 *Volcado de datos*

En función de la tipología de singularidades buscada, así como de las aplicaciones posteriores de los datos obtenidos, el conjunto de la información recabada se resume y vuelca en formatos comerciales, susceptibles de tratamientos posteriores mediante aplicaciones informáticas comerciales.

3 INSPECCIÓN DE LA PRESA DE SAN ESTEBAN

3.1 *Localización*

La presa de San Esteban se encuentra ubicada en el río Sil, en el término municipal de Nogueira de Ramuín. Es una presa de arco-gravedad, con una longitud de coronación de 295 m, una altura de 115 m, con aliviaderos centrales y fue concluida en 1.955.

3.2 *Gestión de mantenimiento*

La empresa Iberdrola mantiene una dedicación especial a la observación del estado y comportamiento de sus numerosas presas, dentro de la cual se contempla la realización de inspecciones directas de manera periódica.

Iberdrola realiza en el año 2001 la primera campaña de Fotogrametría de la presa de San Esteban, recopilando gran cantidad de información para posteriores tratamientos específicos a las patologías detectadas, como fue el caso de tratamientos de filtraciones en juntas frías en los bloques 6i y 7i mediante la inyección de resinas epoxis de alta visco-tixotropía. En el año 2006, se realiza la segunda campaña de inspección por este sistema, logrando obtener un comparativo de las patologías observadas en la primera campaña.

3.3 *Inspección de la presa*

La filmación del paramento se realizó desde las mismas localizaciones empleadas en ambas campañas, de forma de compensar el ángulo de grabación/fotografiado con la curvatura de la presa.

Los bloques susceptibles de filmación fueron los comprendidos entre el 7i y 6d, ambos inclusive, no consiguiendo recabar información de bloques más extremos al encontrarse ocultos tras vegetación o la propia orografía del terreno.

La ejecución del fotografiado se realizó de modo sistemático, en franjas verticales, dejándose un pequeño grado de solape entre sí (tanto con las

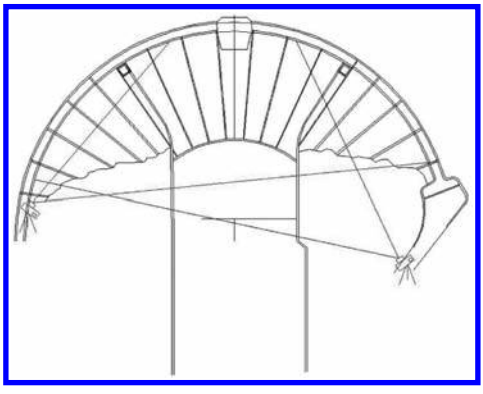


Figura 5. Localizaciones de grabación.

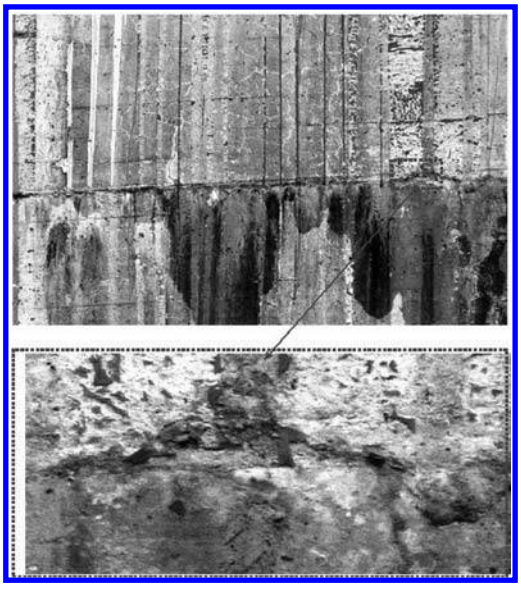


Figura 6. Filtración junta fría.

fotos verticales como horizontales) para permitir posteriormente su unión en gabinete. Para llegar a ello, se dispuso de un trípode motorizado que permite la regulación "in situ" de los parámetros de trabajo, permitiendo su movimiento tanto vertical como horizontal, atendiendo a los condicionantes propios de cada momento.

Dado el empleo de grandes objetivos (Canon 800 mm f/2.6) con teleconvertidores de 2X, el funcionamiento de la cámara se realizo en modo manual para el control de la sensibilidad ISO (pudiéndose ajustar desde 100 hasta 1600 ISO), la velocidad de obturación y la abertura con el grado de luminosidad presente.

En ambas ocasiones, se encontró cierta dificultad de análisis del paramento en los bloques centrales debido a pequeñas fugas presentes en las compuertas.

3.4 *Volcado y tratamiento de datos*

Las imágenes capturadas digitalmente fueron volcadas de forma simultánea a un ordenador portátil de gran capacidad, debido al requerimiento de memoria exigido.

En la campaña del año 2001se obtuvieron cerca de 32.000 fotogramas del video realizado, mientras que en la campaña del año 2006, se realizaron 5400 fotografías de alta resolución con un área aproximada de 3,05 m^2 c/u, facilitando y reduciendo el tiempo de trabajo en gabinete.

Dichas fotografías se dispusieron en forma de mosaico, con gran nivel de detalle.

Es sobre estos montajes sobre los que se realiza la búsqueda de singularidades, siendo una aplicación informática desarrollada por HCC la que va plasmando a tiempo real los defectos sobre plano, extrayendo los datos más significativos (coordenadas, áreas, longitudes, etc.), y creando una base de datos con las particularidades registradas.

3.5 *Tratamiento de los datos*

La información recopilada se gestionó informáticamente, volcándose en forma de documentación gráfica, numérica y estadística.

En lo referido al apartado gráfico, se procedió a segmentar la superficie de la presa en áreas parciales, conformando un grupo de varias docenas de fichas que recogían la información más relevante, acompañadas del montaje fotográfico de la zona correspondiente. Dichas áreas debían coincidir exactamente en la documentación generada en ambas campañas para permitir una fácil interpretación de los datos obtenidos.

Debido a la gran cantidad de información recabada en dichas fichas, entre los cuales, se encontraba la coordenada de todos y cada uno de los defectos localizados, hacia que el tamaño del

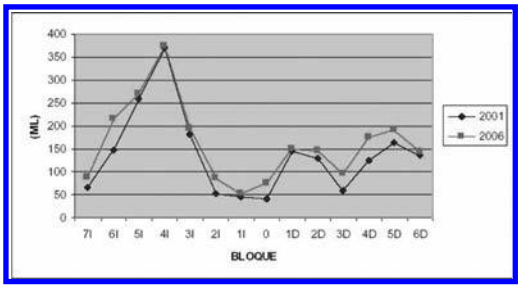
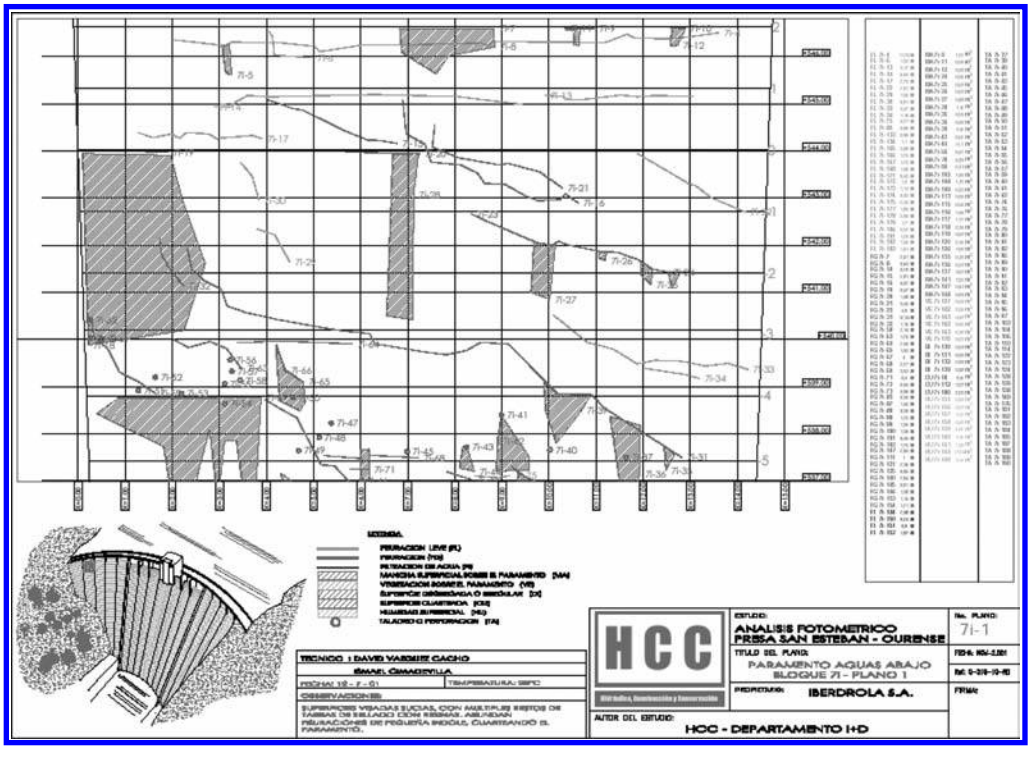


Figura 7. Evolución fisuración leve campañas 2001–2006.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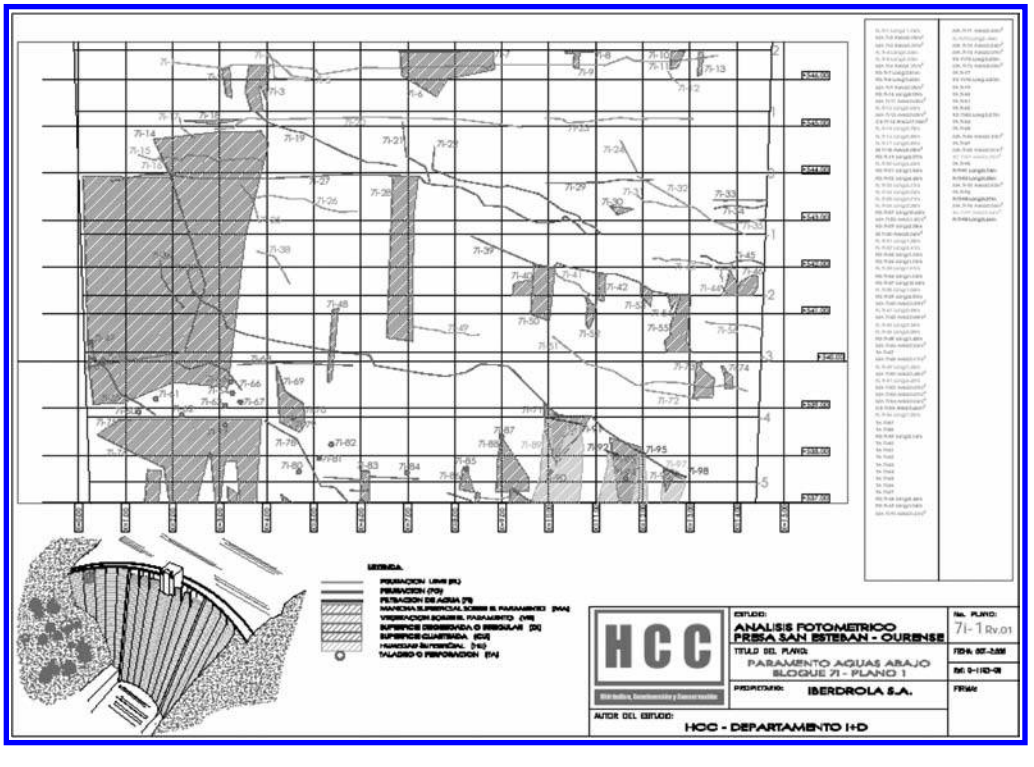
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plano fuese de un tamaño limitado. Se opto por generar un plano de mayor tamaño con coordenadas, de forma de localizar de forma sencilla el defecto en el mismo y en los listados de defectos encontrados, permitiendo una rápida comparativa entre ambas campañas de inspección.

Gracias a dicha información, se realizo el estudio comparativo de cada uno de los defectos por bloque y en datos totales, pudiendo interpretar las variaciones observadas, la evolución de algunos defectos y establecer zonas de posibles actuaciones de mantenimiento.

Para un mejor seguimiento, se montaron planos de conjunto de los distintos bloques, para facilitar visualmente la distribución de los defectos. En lo referente al procesado de la información numérica, se realizó un reparto estadístico de defectos catalogados en forma de gráficas de reparto, dispuestas por bloques y alturas.

3.6 *Conclusiones*

Todos los datos recabados han permitido una observación pormenorizada del paramento de la presa que incluye una cuantificación, clasificación de pequeños defectos existentes y su evolución en el tiempo.

La fotogrametría permite de primera mano establecer y cuantificar los defectos más importantes encontrados en los paramentos verticales de grandes presas y otras estructuras, que por sus dimensiones, no permiten su acceso y estudio.

En resumen, los procedimientos fotogramétricos pueden ser un útil complemento a las inspecciones visuales y a la auscultación tradicional.

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Expansion studies for the Paso Nuevo dam

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ABSTRACT: Among the expansions found in concrete dams, those known as Internal Sulfate Attack (ISA) have not been properly dealt with in the technical literature, as opposed to the Alkali-Aggregate Reactions (AAR). This paper offers a summary of the works carried out for the diagnosis of the Paso Nuevo dam, combining different techniques and emphasizing the importance of an integrated and structural vision of the dam as regards specific aspects. Even though both types of reactions do appear, ISA is the predominant one.

1 INTRODUCTION

Concrete expansion, being a time-dependent phenomenon, is affecting more and more dams since the average age of dams in Spain is increasing. This increase can also be observed in the growing number of scientific-technical publications.

Chemical reactions among the aggregates or between the aggregates and the cement paste are the most common types of concrete expansion; among those, Alkali-Aggregate Reaction (AAR) is the most recurrent and, consequently, the most commonly researched both in Spain [Herrador et al., 2007] and abroad (Mactaquac—Canada, Fontana—United States, Chambon—France [Saouma and Yunping, 2004], Mascarenhas—Brazil [Sabbag, 2003]). Nevertheless, another mechanism which may cause the deterioration of concrete through expansion is the presence of sulfides in the aggregates [Chinchon et al, 1995 and Leea et al, 2005].

In the Paso Nuevo dam, located in the Esera River—a tributary of the Ebro River—in Benasque (Huesca), from 1994 small increases were detected in the leveling heights and the collimation movements of several crest bases [Espinós et al., 2008]. Map cracking was also observed in some areas, as well as diagonal cracking in the gallery close to the left abutment. This is an arch-gravity dam (see Figure 1), built between 1966 and 1969, with a maximum height of 72.60 m and 195 m of crest length.

The aim of this paper is to explain the research works carried out in order to define the diagnosis of the specific behavior registered in this dam. To do so, several complementary techniques have been used: in-depth study of the available historical documentation, visits to the dam, experimental studies and three-dimensional numerical modeling to corroborate the initially stated hypothesis.

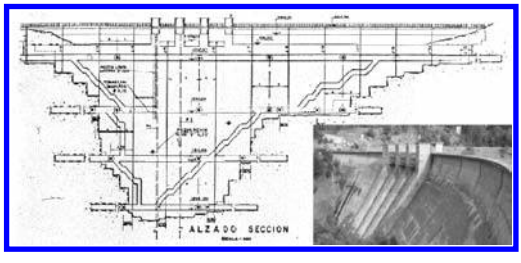

Figure 1. Elevation of the Paso Nuevo dam.

2 DAM BEHAVIOUR

On the basis of the analysis of the movements registered through the auscultation of the Paso Nuevo dam, measured from October 1984 to the moment of submission of this research paper, it can be observed that there exist non-recoverable displacements in collimation, in an upstream direction, of 44 mm, as well as non-recoverable leveling increases of 30 mm, as shown in Figure 2.

After comparing the displacements between the measuring bases, it is ascertained that the blocks which are near the abutments move more, both

in collimation and in leveling, than the central blocks, as can be seen in Figure 3. This behavior is unusual, because the central blocks are normally expected to move more than those close to the abutments, since the former are bigger and more slender than the latter.

In the dam there are also areas with map cracking close to the abutments and in the center of the dam, close to the base, as can be seen in Figure 4, corresponding to the block located underneath the access level of the upper gallery on the left margin. In other areas the concrete looks fine.

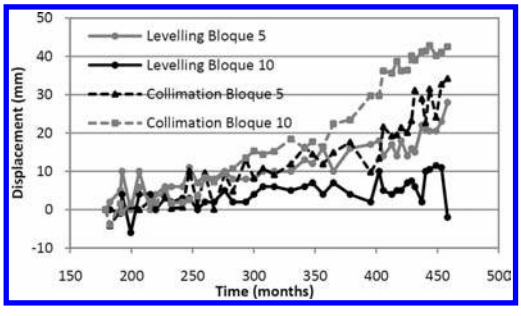

Figure 2. Evolution through time of the crest leveling and collimation movements.

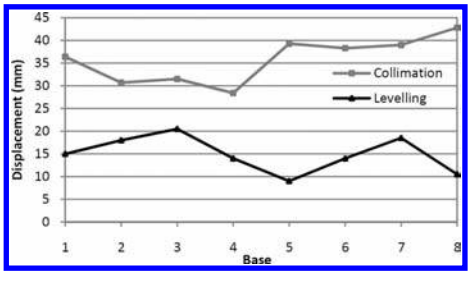

Figure 3. Crest leveling and collimation movements of the several blocks.

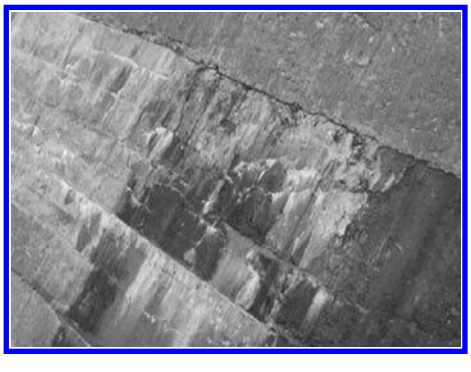

Figure 4. Block cracking in the body of the dam.

3 RESEARCH CARRIED OUT

3.1 *Historical and geological documentation*

In order to come up with a hypothesis for the singular behavior observed, an in-depth study of the historical documentation of the dam was carried out, as well as visual inspections performed during three visits to the structure, and a study of the local geology with maps from the Instituto Geológico y Minero de España (IGME). The dam documents analyzed were:

- Dam book of the Paso Nuevo. It covers from May 13th 1968 to August 10 h 1999, although it also includes an initial summary of the works performed prior to the beginning of the book;
- Building plans—Volume I;
- Reports regarding waterproofing works (1968–1970) and technical inspections (1997, 1999, 2000, 2001 and 2002);
- Data about leveling, collimation, pendulums, manometers and joint displacements from 1984 to 2009;
- Water analysis (1999, 2000 and 2001).

On the basis of the dam book, the map of concrete-pouring was re-established (Figure 5), including some blocks of which no information is available, since they were built prior to the summer of 1968.

The results obtained through the study of the geological maps from the Instituto Geológico y Minero de España (1971) indicate that the area near the dam shows a geology notably changing within short distances, corresponding to the Quaternary, Devonian and Silurian periods. The above-mentioned area is specifically made up of: indifferent alluvium, calcite and slate, and metamorphic slates, respectively, which will later be experimentally contrasted.

3.2 *Visual inspection*

During the visits to the dam it was observed that the areas with map cracking were located in blocks near the abutments and in the central blocks under

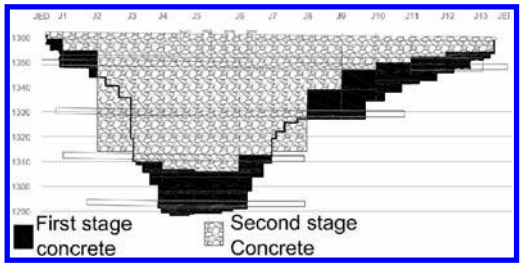

Figure 5. Concrete-pouring stages in the dam.

the 1310.00 m benchmark, corresponding to the blocks where concrete was poured before 1968, about which no recorded data remains. Diagonal cracking was also observed in the gallery under the 1350.00 m benchmark near joint 11 (As can be seen in Figure 6).

During the visits it was also observed that there were two quarry faces (Figure 7): a small one corresponding to the beginning of the concrete-pouring works (1st stage), since it was the closest one to the concrete-mixing plant; and a bigger one which was probably used in most of the dam (2nd stage).

3.3 *Differences between project and actual site*

Having compared the information obtained through the building plans with that obtained through the visits to the dam, it was possible to observe some differences between the projected structure and the actually built structure, such as, for instance, the existence of concrete sealing blocks in the right abutment (upstream and downstream faces) and in the left abutment (downstream face).

Figure 6. Diagonal cracking in the 1350 m benchmark gallery, close to joint 11.

Figure 7. Quarry faces used during concrete-pouring in the dam.

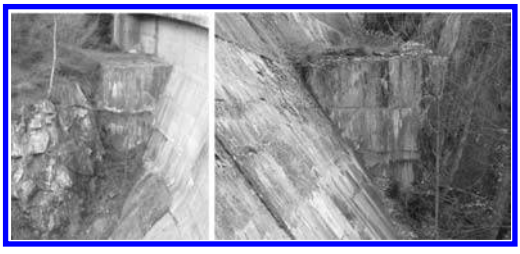

Figure 8. Sealing blocks in the abutments (downstream).

Figure 8. shows in detail the concrete sealing blocks from the Paso Nuevo dam (downstream face and left abutment).

As can be seen, they are full of map cracking, which is characteristic of concrete expansion. Owing to their location, an expansion in them might cause a thrust in the blocks which are close to the abutments upstream, which in turn might cause the displacements of the collimation points in the abutments, as well as torsional internal forces in the structure [Espinós et al., 2008].

3.4 *Cores and their results*

In order to make a diagnosis, a core extraction campaign was devised, with cores to be extracted from different blocks (affected or unaffected by expansions) and from different areas of those blocks (face, gallery, foundation and crest). The aim of this was to identify the possible reactions which might degrade the concrete and affect its durability, that is, Alkali-Aggregate Reactions (AAR) and Internal Sulfate Attack (ISA), even though the previous studies made it possible to put forward a hypothesis beforehand. With this purpose, an experimental campaign was prepared, including the following types of tests:

- Verification, by means of selective staining, of AAR and ISA reactions in the surface of the cores;
- Visual analysis and analysis by means of Optical Microscopy (OM);
- Instrumental analyses: X-ray Diffraction and Fluorescence (XRD and XRF) and Scanning Electron Microscopy (SEM), performed on the most relevant samples.

The results from core H4 are shown as an example. Core H4 was located in the gallery with 1330.50 benchmark in block 9, downstream direction. The concrete belongs to the first stage and neither apparent cracking nor signs of oxidation are observed. The core shows aggregates with wide diameters (Ømax > 12 cm).

After carrying out the inspection by means of OM, strong traces of ISA with abundant ettringite

Table 1. Comparison between relative slenderness and the relative collimation movement.

Base	Block	Height (h) m	Base (b) m	j = h²/b	(C) mm	j/j8	C/C8
1	3	52,68	38,08	72,88	35,0	2,57	0,82
2	4	63,43	47,07	85,48	29,4	3,01	0,69
3	5	57,17	42,10	77,63	34,2	2,73	0,80
4	6	45,36	32,47	63,37	31,3	2,23	0,73
5	7	33,36	23,57	47,22	38,2	1,66	0,89
6	8	28,46	20,01	40,48	38,4	1,42	0,90
7	9	24,91	17,35	35,76	39,2	1,26	0,92
8	10	19,12	12,90	28,34	42,5	1,00	1,00

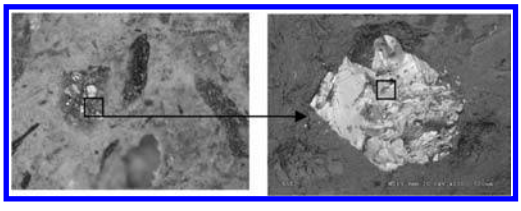

Figure 9. Presence of iron sulfides.

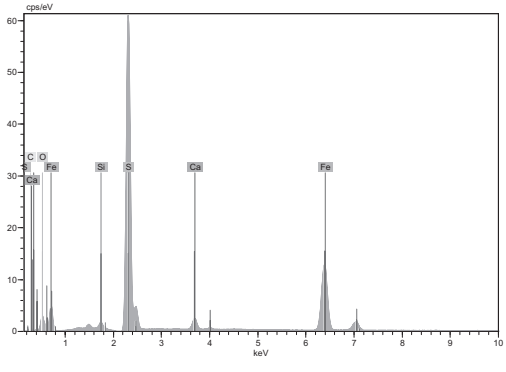

Figure 10. Results of the micro-analysis by means of SEM.

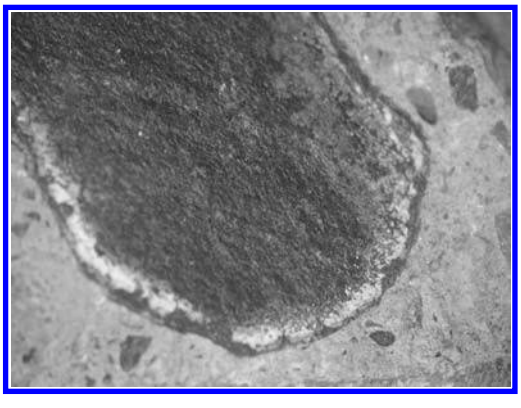

Figure 11. Aureoles characteristic of AAR.

are found. In the ettringite, the possible presence of iron sulfides can also be noticed, contrasted by means of SEM (Figure 9). The results from the micro-analysis of the marked area in Figure 9 are shown in Figure 10.

Table 2 shows the quantification of the elements found in sample H4-19. From the relation or proportion between S and Fe, it seems that they are pyrite crystals. In other samples, pyrrhotite and even AAR gels were found (see Figure 11), although ISA reactions are predominant.

Likewise, in some of the rock samples there was a possible fossilization of ettringite crystals produced when they were covered by AAR gels after their formation. Usually, the reaction kinetics of ISA are faster than those of AAR, which can be of great help for the diagnosis.

It is worth remembering that secondary ettringite is an expansive product which appears after the reaction of the Calcium Aluminates (C_3A) present in cement with the sulfate ions coming, in this case, from the oxidation of the iron composites present in aggregates (pyrite and pyrrhotite), as shown in the following reaction.

$3SO_4^{2-} + C_3A + Ca(OH)_2 + 32H_2O \rightarrow 3CaO.Al_2O_3.3CaSO_4.32H_2O$

Sulfates + aluminates + portlandite + water → ettringite

4 BEHAVIOR HYPOTHESIS

4.1 *Internal sulfate attack*

After the analysis of the data and the visual inspection, the doubt remains: why two quarry faces? One of the possibilities is that perhaps they began pouring concrete with aggregates extracted from the first quarry and, after verifying possible problems with the aggregates, they might have opened the second quarry face. This hypothesis was reinforced upon observation of the local geology, which shows important variations within short distances.

Another interesting aspect is that, when comparing the locations of the cases of map

Table 2. H4-19 sample: results from the quantitative analysis.

Element	Net	unn. C [wt.-%]	norm. C [wt.-%]	Atom. C [at.-%]	Error [%]
Carbon	2823	2.48	2.06	5.78	0.6
Silicon	7323	0.91	0.75	0.90	0.1
Sulfur	484383	53.01	43.90	46.18	1.9
Calcium	19836	2.54	2.10	1.77	0.1
Iron	164954	50.23	41.59	25.12	1.4
Oxygen	17000	11.59	9.60	20.24	2.1

cracking with the concrete-pouring map, it becomes clear that all those cases of cracking are located in the area corresponding to the concrete-pouring carried out during the first stage of the building works, which is not recorded in the dam book (Figure 5). In the documentation there is no information regarding when the change of quarry took place, but it is reasonable to think that the initial concrete-pouring which is not reflected in the book corresponds mainly to the first quarry face.

As a first hypothesis, it was proposed that the dam might show expansion reactions caused by the presence of iron sulfides in the aggregates used in the concrete. This hypothesis was reinforced by a subsequent analysis of the problems encountered in other dams in nearby locations, and by a visit for an in-situ inspection. During this visit, stains on the surface of concrete were observed. Those stains showed a dark-brown color (possible oxidation of the iron sulfides present in the aggregates); however, it was not possible to conclude whether those sulfides came from the pyrite (FeS_2) and/orpyrrhotite (Fe_{1-x} S) minerals [Casanova et al. 1996].

Internal Sulfate Attack (ISA) may occur in two different stages within concrete: in a primary reaction due to the oxidation of iron sulfide (pyrites and/or pyrrhotites) present in the aggregate, and in a secondary reaction when the sulfides and ions of hydrogen released in the pore of the solution react with the Portland cement paste and form additional expansions in the phases: plaster, monosulfo-aluminate and, finally, ettringite [Araujo, G., 2008]. Consequently, it seems reasonable to think that it is a matter of internal sulfate attack in the aggregates from the first quarry, which seems to be correct since:

- It is the blocks with a higher leveling height that present a great amount of concrete from the first quarry (block 5 between J5 and J6) (Figure 12);
- The block with the lowest leveling height is the one which presents less concrete from the first quarry (block 7 between J7 and J8);
- Collimation movements are greater near the abutments because in that area there is more concrete from the first quarry.

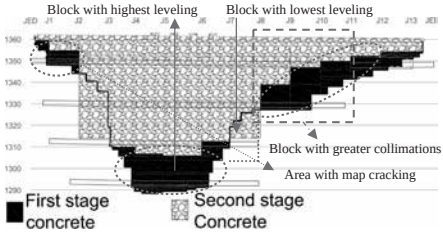

Figure 12. Correlation between the concrete-pouring map and the displacements in the dam.

If the expansive reactions occur predominantly in the concrete with aggregates from the first quarry, as defined by the hypothesis, the phenomenon is not generalized. This affirmation was verified after establishing a parallelism between the displacements obtained and the slenderness and height of the blocks.

This verification of the relationship between the slenderness of the blocks and their relative displacements was proposed on the basis of the research studies carried out by Martínez et al., (1991), where the possible causes for the anomalous behavior of the Presa de Graus (Spain) were studied. In that research it was possible to verify that the different displacements of each block are the result of a similar action in all of them, even though they have a different geometry. The phenomenon which has been described as expansive affected the entire dam.

By applying the same methodology to the Paso Nuevo dam, the results shown in Table 1 were obtained. In the case of the Graus dam, there existed a correlation between slenderness/collimation movements (c) and height/leveling; but in the case of Paso Nuevo there is no such correspondence, which is coherent with the fact that the problem is not general, since it only affects a specific set of blocks, especially those with concrete containing aggregates from the first quarry face.

4.2 *Torsion in the dam*

As regards the diagonal cracking found in the gallery of 1350 m benchmark, they have a format

generally decurrent regarding torsion effects, but what could cause torsion in the dam?.

A hypothesis which explains the cracking is the fact that, as can be observed in the collimation graphs, the blocks close to the abutments show greater displacements towards the upstream face than the central blocks do, because of the difference in displacements.

The instances of cracking shown in Figure 5 are only located in the left abutment, which might be due to the fact that the shape of the right abutment offers resistance to displacement in upstream direction, and that reduces the effect in relation with the left one, and also to the fact that the amount of concrete from the first quarry is greater in the right abutment.

5 3D MODELING OF THE DAM

5.1 *Generation of the geometry and the mesh*

The 3D representation of the geometry of the dam and the soil was carried out on the basis of the building plans, the data from the chronogram for concrete-pouring included in the dam book, and some aspects not referenced in the available documentation but detected during the visits (such as the change in direction of the gallery of the 1350 benchmark in the area close to the left abutment, and the existence of concrete sealing blocks).

Figures 13 and 14 show views of the geometry of the dam, from the downstream and upstream faces, respectively, where the soil is indicated in green, the different dam blocks appear in grey and red stands for the concrete sealing blocks.

In those figures, several aspects of the geometry have been simplified with the aim of reducing the degrees of freedom of the problem during the process of mesh generation (which are in themselves high in a 3D representation). In order to do so, the exposed surface of the terrain in areas far away from the dam, as well as the dam/soil contact of the more central blocks (from J3 to J11), have been simplified. The dam/soil contact of the rest of the blocks has been kept approximately in agreement with the information from the plans, since they correspond to the area where the possible torsion effect on the dam is to be studied. Only the upper gallery has been represented (1350 benchmark), which corresponds to the area of interest where instances of diagonal cracking are found, near the left abutment.

In the central area of the dam (from J3 to J11), the geometry of the gallery has been simplified into a rectangular shape, whereas its original geometry is maintained in the rest of the blocks, where the above-mentioned cracking is observed. The change in direction of the gallery at 1350 benchmark, in the area close to the left abutment, has also been taken into account. The dam concrete has been represented establishing a difference between the areas corresponding to the first or to the second quarry, according to the information from the concrete-pouring plan, which can be seen in Figure 5. The concrete sealing blocks observed during the visits have also been discretized; those blocks do not appear in the building plans of the dam, but they are thought to have a significant influence over its mechanical behavior.

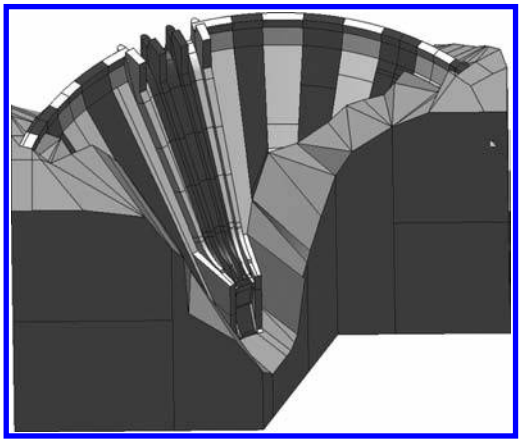

Figure 13. View of the dam geometry from downstream.

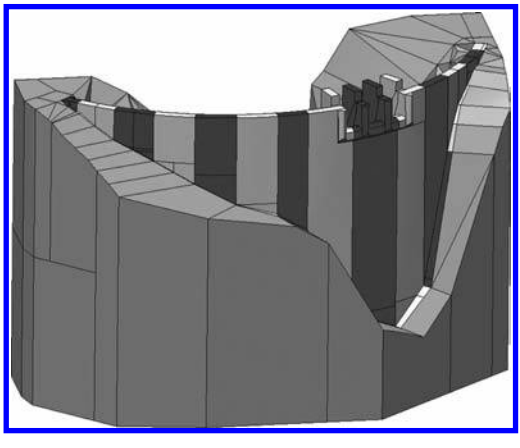

Figure 14. View of the dam geometry from upstream.

Joints plans were later incorporated into the original geometry in the contact of the dam with the soil and in vertical plans between the blocks of the dam. The initial calculations highlighted the need to incorporate the effect of the joints between the blocks, and their considerable influence will be seen in the results shown below.

Beginning with the geometry, different meshes can be generated by varying the density of finite

elements depending on different aspects, such as areas with a higher research interest, strain gradients due to the characteristics of the expansion, etc. For the initial examples shown in this paper, a mesh without a high density of elements was generated, in order to limit the number of nodes and, therefore, the calculus dimensions of the problem, since in the first place the purpose is to evaluate and adjust the global displacements of the structure. The generated mesh is made up of 64326 nodes, 268669 linear tetrahedrons as continuum medium elements, and 16359 zero-thickness interface elements.

For the contour conditions, vertical displacement (z) has been restricted in the low horizontal plane of the soil, as well as the displacements in x and y directions in the vertical planes which form the contour of the soil.

For both the generation of the geometry and the mesh of the finite elements, and for the subsequent graphic representation of the results from the calculations, the program used was GID, developed at the CIMNE, UPC. On the other hand, the calculations were carried out by means of the finite elements code called DRAC, developed at the Department of Geotechnical Engineering and Geo-Sciences from the ETSECCPB-UPC.

5.2 *Initial results*

For the concrete and the rock, the following parameters were adopted: elasticity module (E) = 20 GPa and 70 GPa, and Poisson coefficient (ν) = 0.2 for both.

For the interface elements, a linear elastic law was used. In the dam/soil contact, very high values are adopted for the rigidity coefficients ($Kn = Kt = 10^9$ KN/m^3), in order to simulate a very solidary behavior between the dam and the soil (equivalent to the absence of joints). For the joints between dam blocks, a very high normal elastic constant was adopted ($Kn = 10^9$ KN/m^3), while for the tangential elastic constant three cases have been assumed: one with a very high Kt ($Kn = 10^9$ KN/m^3), which would correspond to the solidary behavior; another one with a low value ($Kn = 10^2$ KN/m^3), which implies a low drag effect between blocks; and a third one with an intermediate value ($Kn = 10^5$ KN/m^3).

In the calculation examples, a comparison is made between the displacements obtained in the mesh nodes which are closer to the real location of the reference bases in the crest, according to the diagram shown in Figure 15 (the corresponding block number appears between brackets).

In the first place, calculation results were obtained for the actions with self-weight and hydrostatic pressure. In this case, the crest displacements show a maximum value of 0.46 mm

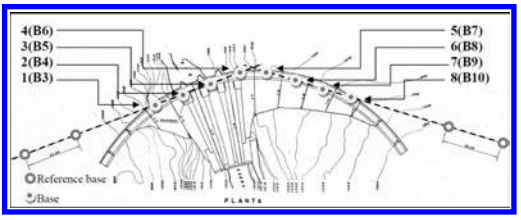

Figure 15. Location of the reference bases in the dam crest.

in the vertical direction (leveling) and 2.00 mm in the horizontal direction (collimation) for the case of intermediate Kt. These values are much lower than the displacements registered in the dam, mainly brought about by some other cause, presumably due to expansive reactions. Likewise, the main stresses obtained have also been low.

In order to incorporate the expansive actions, it has been considered that the whole expansion is developed only in the concrete coming from the first quarry face and in the concrete sealing blocks. First of all, with the aim of evaluating the order of magnitude of the imposed strains which would justify the displacements currently registered, it has been determined that a constant imposed strain of 1.5 mm/m makes it possible to approximately adjust the vertical displacement registered in base 7.

In previous studies of gravity dams which showed expansive effects due to Internal Sulphate Attack (ISA), such as Graus and Tavascan, it was verified that the effect of ISA has a greater intensity on areas near the downstream face, which in comparison with the upstream face is exposed to a greater concentration of oxygen; thus, the tendency is for the dam blocks to move towards the upstream face (Aguado et al. 1998).

In order to be able to take this situation into account in a simplified manner, it has been considered that the concrete from the first quarry face shows a distribution of strains which vary lineally, with a 4 mm/m value in the downstream face and a 1 mm/m value in the upstream face. Actually, the distribution of the acting strains will be non-linear, but the linear hypothesis makes it possible to conveniently adjust the displacements (obviously not the stresses), which is the purpose of these preliminary studies. In the case of the concrete sealing blocks, since they show a large surface exposed to the air, a constant strain of 4 mm/m has been assumed. These strain values make it possible to approximately reproduce the levelling and collimation movements of base 7.

Figures 16 and 17 show the vertical and horizontal displacements obtained in the crest, calculated for the three values of Kt considered. The registers

of displacements measured in the year 2006 for each base of the dam are also included.

The great influence of the constant Kt can be observed in both figures. When little cutting interaction between the joints is simulated (Kt = 10^2 KN/m^3), the blocks with a greater amount of reactive concrete move much more than the other ones, whereas when a high interaction between blocks (Kt = 10^9 KN/m^3) is imposed, a strong drag effect is created, which tends to make the value of the displacements uniform.

Even though adjustments still have to be made as regards the imposed strains, through these figures it can be seen that a Kt with an intermediate value would be the one which best reproduces qualitatively the tendency observed in the real measures (grey continuous line). Figure 16 shows that the intermediate Kt reproduces qualitatively well the tendency observed in the real measurements, with a greater vertical displacement in the bases 3 (between J4 and J5) and 7 (between J9 and J10).

On the other hand, Figure 17 shows that the intermediate Kt reproduces adequately the measured tendency of greater collimation movements in

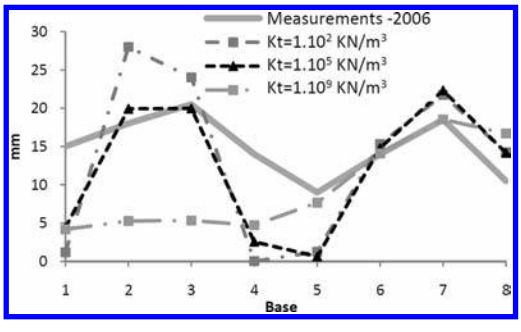

Figure 16. Vertical displacements calculated for different values of Kt, along with results from measurements in the dam.

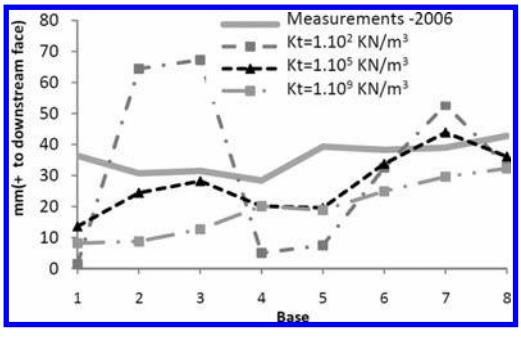

Figure 17. Horizontal displacements calculated for different values of Kt, along with results from measurements in the dam.

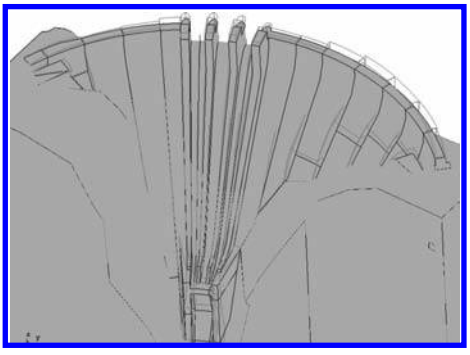

Figure 18. Deformed configuration of the dam in red lines (× 50). In green, non-deformed geometry.

the blocks of the left half which are closer to the abutment (bases 6, 7 and 8).

Figure 18 shows the deformed configuration of the dam in the case of the intermediate Kt. As can be seen, the blocks on the left half of the dam (blocks on the right in Figure 18) have a greater tendency to move horizontally than the rest of the blocks. It can also be observed that the expansion of the concrete sealing blocks tends to increase the collimation movements of the blocks which are in contact with them.

6 CONCLUSIONS

From the results gathered in the present paper, the following main conclusions can be drawn:

- In the results obtained from the analyses of the concrete cores, it was found that secondary ettringite was present in most of the cores analyzed, its presence being in some of them very pronounced. This is a consequence of the reaction of the calcium aluminates present in the cement with the sulfate ions coming from the oxidation of the aggregates existing in concrete.
- The 3D model developed for the structural calculation of the Paso Nuevo dam is a powerful tool for analyzing and evaluating the anomalous behavior detected in the previously mentioned dam. The results obtained up to now, preliminary though they are, highlight that the hypothesis of an expansive action affecting only the concrete from the first quarry face generates vertical and horizontal displacements in spots of the mesh close to the reference bases in the crest, which coincide adequately with the displacements measured in the real structure.
- The development of the study has revealed the great importance of producing and filing for future the adequate supporting building documents in order to get a fast and accurate diagnosis.

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Reacción álcali-sílice. Técnicas rápidas para la evaluación de áridos reactivos

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RESUMEN: La reacción álcali—sílice es una patología expansiva característica de presas de hormigón. En la detección de áridos reactivos, las metodologías que se están desarrollando y adoptando actualmente tienden a emplear ensayos de larga duración para poder abarcar todos los tipos de áridos reactivos, incluyendo los áridos de reacción lenta.

En esta comunicación se describen otros ensayos desarrollados en una investigación sobre la reacción álcali-sílice llevada a cabo en el Laboratorio Central de Estructuras y Materiales del CEDEX que, de forma rápida, permiten obtener resultados para descartar la reactividad de un árido en un corto periodo de tiempo, máximo de tres días. Estos ensayos permiten valorar, cuando se seleccionan los materiales componentes para el hormigón de una presa de nueva construcción u otra obra hidráulica, varias canteras de posible utilización de una forma rápida.

Así, y basándose en el procedimiento del ensayo Gel-Pat, en el Laboratorio Central de Estructuras y Materiales del CEDEX se ha desarrollado un nuevo ensayo para la detección de áridos e identificación de componentes reactivos en un periodo de tres días. De forma sencilla y rápida se clasifican los áridos como potencialmente reactivos o inocuos. También se ha aplicado la difracción de rayos X, empleando la medida de los dominios cristalinos, para diferenciar formas reactivas e inocuas de cuarzo. Por último, conocidas las limitaciones reflejadas en la bibliografía para los ensayos químicos, se ha empleado la medida de sílice soluble para diferenciar en cuarcitas, cuarzoarenitas y arenas silíceas, áridos reactivos de inocuos.

ABSTRACT: Alkali silica reaction is an expansive pathology of concrete frequent in dams. Methodologies developed nowadays to detect reactive aggregates are too long in order to detect also slow reactive aggregates.

This paper describes other rapid tests developed in Laboratorio Central de Estructuras y Materiales of CEDEX to accept or reject aggregates (3 days maximum). These tests are useful to select the aggregates among several options for construction of new dams or hydraulic works in a short period of time.

A new test based on Gel-Pat Test has been developed for detecting reactive aggregates and identify the reactive components in three days. Modified Gel Pat test becomes easy and quick to classify aggregates as innocuous or potentially reactive. Also, domain size measured with X Ray diffraction has been applied to detect reactive forms of quartz. Lastly, after analyzing limitations of chemical tests for detection of reactive aggregates, soluble silica has been used to classify quartzites, quartzarenites, and siliceous sands as reactive or innocuous.

1 INTRODUCCIÓN

La Reacción Álcali-Sílice (RAS) es aquella que se produce entre formas reactivas de sílice presentes en los áridos y los álcalis de la disolución intersticial del hormigón. Los productos de reacción formados, al embeber agua, aumentan su volumen provocando el deterioro del hormigón (CNEGP 1991).

Debido a la necesidad de agua para que se produzca la expansión del hormigón (ACI 1998), la reacción álcali-sílice es una patología muy relacionada con las presas, y en general, con las obras hidráulicas.

Para prevenir la aparición de expansiones en obras hidráulicas por RAS es fundamental la elección de áridos inocuos para su construcción.

Existen gran cantidad de ensayos para la identificación de áridos reactivos, pero todos presentan limitaciones. Para evitar fallos en el estudio de áridos reactivos, cada país, a partir del conocimiento de sus áridos reactivos, ha creado una metodología de estudio, ordenando los ensayos y fijando sus límites de forma que sea válida para la detección

de sus áridos reactivos. (CEN 1995; EUROPEAN COMMISION 1996; Nixon 2004; Chak 2005; Hönnum 2006).

De la recopilación de metodologías de distintos países (Chak 2005; Hönnum 2006), se deduce que la propuesta más adoptada por los diferentes países es la que se muestra en la Figura 1.

La petrografía se emplea como ensayo inicial para evaluar si el árido podría ser reactivo, en cuyo caso se deben continuar realizando ensayos para definir su grado de reactividad (Nixon 2004).

En tal caso, el ensayo determinante para definir la reactividad es el ensayo de prismas de hormigón (Rilem T.C. 106 2000; Thomas 2006), que en su forma acelerada tiene una duración mínima de 6 meses. El ensayo acelerado de probetas de mortero se emplearía como anticipo a los resultados posteriores de prismas de hormigón.

El mayor inconveniente que presenta el modelo adoptado internacionalmente es su larga duración.

En esta comunicación se proponen otros ensayos que, de forma rápida, permiten obtener una primera aproximación al árido para descartar su reactividad en un corto periodo de tiempo, máximo de tres días. Estos ensayos permiten valorar, cuando se seleccionan los materiales componentes para el hormigón de una presa de nueva construcción u otra obra hidráulica, varias canteras de posible utilización de una forma rápida.

Se han analizado tres técnicas: el ensayo Gel-Pat, la difracción de rayos X y el ensayo químico. De cada ensayo se han recogido las limitaciones y ventajas que presentan, así como los tipos de áridos en los que pueden ser de aplicación estas técnicas. A partir de este análisis se ha definido un nuevo ensayo basado en el ensayo Gel-Pat y se ha empleado la difracción de rayos X y la sílice soluble del árido para caracterizar áridos reactivos.

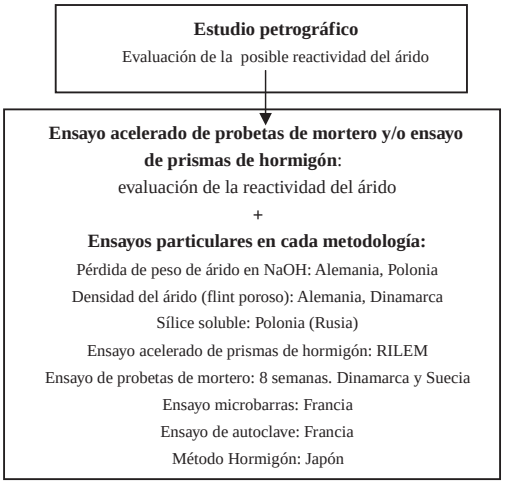


Figura 1. Metodología más generalizada para la detección de áridos reactivos.

2 GEL PAT MODIFICADO

2.1 *Introducción*

El Ensayo Gel-Pat aparece en el Anexo B (informativo) de la norma "Interpretación del estudio petrográfico para reactividad álcali-árido" (BRITISH STANDARD BS 7943 1999). Este ensayo proporciona un método para la detección de formas de sílice opalina u otros compuestos de sílice altamente reactivos con los alcalinos, haciendo que se formen geles en la superficie de éstos al reaccionar con los álcalis.

Una fracción de árido silíceo, embebida en un disco de pasta de cemento, se introduce en una disolución alcalina. Durante su almacenamiento a 20ºC, los discos se examinan de forma periódica para observar la aparición de cualquier gel. El crecimiento de geles en la superficie de los áridos durante los 14 días del ensayo será indicativos de la probable existencia de sílice opalina u otras formas de sílice altamente reactivas.

El límite de detección, en función del número de partículas ensayadas se ha estimado entre el 3% (100 partículas) y 0,1% (2000 partículas).

Algunas de las limitaciones en la aplicación de este ensayo se exponen a continuación:

– Este ensayo está indicado para la detección de ópalo pero no ha sido eficaz para la detección de otros componentes reactivos como el cuarzo micro y criptocristalino presentes en áridos silíceos reactivos. Estos fallos pueden deberse a que la reactividad del ópalo es muy superior a la del cuarzo micro y criptocristalino ya que se trata de una forma amorfa e hidratada de sílice y el

Fotografía 1. Ensayo Gel-Pat (BS-7943).

cuarzo, aunque sea reactivo por su tamaño de grano, es una estructura cristalina y, por lo tanto, termodinámicamente más estable y menos reactiva que el ópalo.
– Se ha observado que la superficie del disco en la que se encuentran los áridos en el ensayo Gel-Pat es muy compacta. La inexistencia de huecos dificulta el crecimiento de geles.

2.2 *Ensayo Gel-Pat modificado para la detección de áridos potencialmente reactivos*

Por todo lo expuesto anteriormente, en el Laboratorio Central de Estructuras y Materiales del CEDEX, se ha creado un nuevo ensayo basado en el Gel Pat para conseguir que sea positivo con rocas que contengan cualquier componente reactivo, reducir su duración y, en el caso de que sea positivo, poder extraer con facilidad los áridos que han originado los geles e identificar el componente reactivo. Para ello se ha aumentado la temperatura de ensayo a 80ºC y modificado la forma de fabricar las pastillas para aumentar el número de huecos y así facilitar el crecimiento de los geles.

Las ventajas obtenidas con estas modificaciones se exponen a continuación:

– Temperatura.

Como en todas las reacciones químicas, la temperatura es un factor determinante en la evolución de RAS. El avance de la reacción aumenta según la temperatura asciende. Al mismo tiempo, los geles a altas temperaturas disminuyen la viscosidad y aumentan su capacidad para acumularse en los huecos del interior del hormigón. Por tanto, al aumentar la temperatura se acelera la reacción acortando el tiempo de duración del ensayo y se facilita la aparición de geles en huecos y fisuras.

– Creación de huecos.

Las probetas se fabrican de forma que se formen entre las partículas de árido gran cantidad de huecos. De esta forma se favorece que los geles se acumulen libremente en estas zonas.

– Separación de partículas reactivas.

Se pueden identificar las partículas que han generado los geles y extraerlas para su estudio petrográfico. De esta forma se identifica de forma rápida el componente reactivo que ha originado la reactividad.

2.3 *Aplicación a dos áridos reactivos*

El ensayo ha sido probado en 2 áridos reactivos españoles de los que es conocida su reactividad, por haber sido extraídos del hormigón de presas españolas afectadas por RAS. El primero de ellos (Árido A) es de reacción rápida por su contenido en cuarzo micro y criptocristalino y un segundo (Árido B) de reacción lenta debida al cuarzo altamente deformado y fisurado (tablas 1 y 2).

Tras los tres días de ensayo, los 2 áridos reactivos españoles han generado gran cantidad de geles distribuidos en su superficie (fotografías 2 y 3).

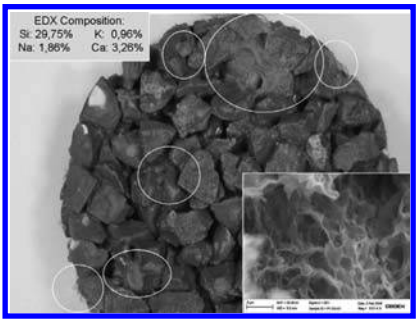
Tabla 1. Descripción petrográfica de los aridos reactivos.

Árido A. Cuarzoarenita. Homogénea	Cuarzo. (90–99%) en granos de cuarzo principalmente. También aparecen granos de sílex	Opacos, óxidos de hierro, filosilicatos, moscovita, turmalinas y argilomicáceas.
Árido B. Granito de dos micas	Cuarzo: 25–35% Feldespato potásico: 20–40% Plagioclasa: 25–35% Biotita: 8–9% Moscovita: 1–2%	

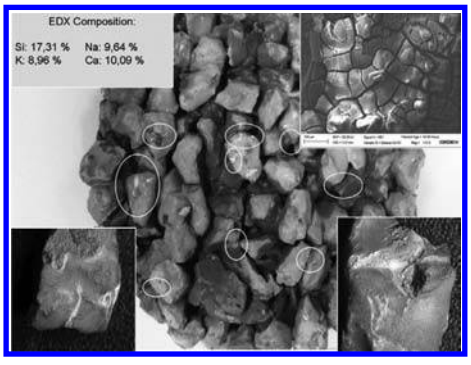
Tabla 2. Componentes reactivos en los áridos.

Árido	Fracción	QFG	QMC	QCC
A	2–4	32,3	20,9	2,2
	<2	18,8	12,9	1,2
	Extinción ondulante en el total de granos de cuarzo identificados en cada fracción			
B	2–4	95% (27°)		
	<2	95% (26°)		

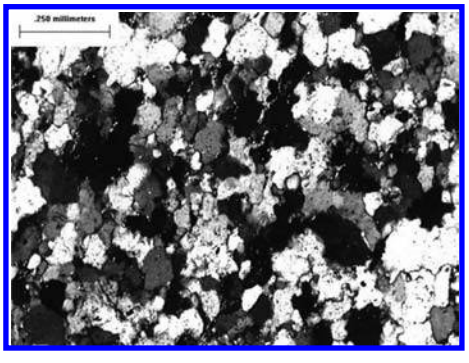
QFG: Cuarzo de grano fino.
QMC: Cuarzo microcristalino.
QCC: Cuarzo criptocristalino.



Fotografía 2. Árido A. Formación de gran número de geles en la superficie de los áridos. Identificación de los geles formados como productos de reacción álcali-sílice por MEB-EDX.



Fotografía 3. Árido B. Formación de gran número de geles en la superfície de los áridos. Detalle de los geles formados sobre uno de los áridos reactivos e identificación como productos de reacción álcali-sílice por MEB-EDX.



Fotografía 4. Árido A. Identificación de cuarzo microcristalino como componente reactivo.

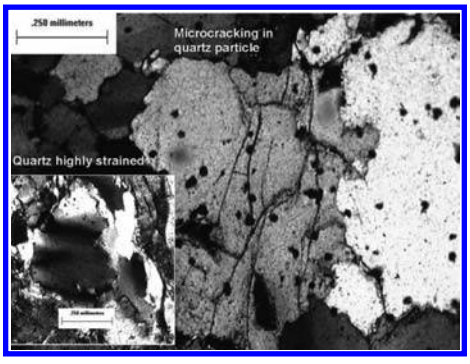
Estos geles son fácilmente identificables visualmente. En todos los casos se ha confirmado su origen por MEB-EDX, siendo productos de reacción álcali-sílice.

Se han seleccionado partículas con geles cercanos para su caracterización petrográfica. Los resultados obtenidos son:

– Árido A. Realización de dos láminas delgadas para la caracterización petrográfica de 9 partículas. 4 de las 9 contienen cuarzo micro y criptocristalino (fotografía 4).
– Árido B. Realización de una lámina delgada para caracterizar 4 partículas de árido que tenían en su superficie algún depósito. El componente reactivo es el cuarzo deformado, recristalizado y microfisurado, identificado en las partículas con gel en su superficie (fotografía 5).

De los resultados obtenidos con estos dos áridos reactivos se puede deducir que:

– El ensayo Gel-Pat modificado propuesto permite, de forma muy rápida (3 días) y sencilla (visualmente), detectar en los áridos la presencia de componentes reactivos de reacción rápida y de reacción lenta.
– Realizando un número muy limitado de láminas delgadas se puede identificar el componente reactivo.
– El ensayo es rápido y sencillo, pero su resultado es cualitativo. Clasifica al árido como inocuo o potencialmente reactivo, pero no cuantifica el grado de reactividad en este último caso. El ensayo ayuda a seleccionar áridos inocuos (sin aparición de geles en el transcurso del ensayo).



Fotografía 5. Árido B. Identificación de partícula de la pastilla con cuarzo microfisurado y altamente deformado como componentes reactivos.

3 ENSAYO QUÍMICO. MEDIDA DE SÍLICE SOLUBLE

3.1 *Introducción*

De los ensayos existentes para la detección de áridos reactivos, los ensayos químicos presentan la ventaja de ser ensayos rápidos, ya que proporcionan resultados en 24 horas.

Para la realización del ensayo químico ASTM (ASTM C 289 2001) se toman 3 muestras de 25 g de la fracción 0,150–0,300 mm. La muestra de árido es sumergida en una solución 1 N de hidróxido de sodio a 80ºC, midiendo en el filtrado, después de 24 h, las concentraciones de sílice disuelta y la reducción de alcalinidad por absorción ultravioleta y valoración ácido-base, respectivamente.

Un mayor contenido de sílice disuelta en la disolución indica una mayor reactividad del árido.

Los ensayos químicos presentan inconvenientes que han quedado recogidos en la bibliografía. Por ejemplo, la norma ASTM (ASTM C 289 2001) especifica que el ensayo no predice áridos de reacción lenta, producida por áridos que contienen cuarzo deformado o microcristalino, o áridos que contienen metagrauwacas, metacuarcitas o áridos

similares. Además, sólo deberían utilizarse como método secundario, y sometidos a unas limitaciones de uso estrictamente definidas (Rilem T.C. 106 2000).

En la bibliografía se han planteado límites para el ensayo químico, teniendo en cuenta sólo el valor de sílice soluble. Así, hay fuentes que fijan el límite entre reactivo e inocuo en 100 mmol/l, aunque otras bajan el límite hasta 50 mmol/l (Shayan 1986). También se han fijado límites en función del tipo de roca reactiva estudiado (Grattan-Bellew 1983) comprendidos entre 50 y 200 mmol/l.

Los criterios fijados deben ser propios de cada país ya que, por ejemplo, el límite de 50 mmol/l para el flint danés, no es válido para graveras con flint en Gran Bretaña, que han resultado inocuas con valores de Sc mayores de 100 mmol/l.

Las últimas publicaciones sobre ensayos químicos correlacionan la sílice soluble con la expansión en prismas de hormigón de los áridos aplicando dos correcciones: el contenido en aluminio soluble en disolución alcalina y la porosidad de éstos (Hünger 2008):

– La porosidad tiene influencia en valores comprendidos entre el 2% y 5% ya que disminuye la expansión que produce el árido, por lo que aumenta el límite reactivo inocuo
– El contenido en aluminio disminuye la capacidad expansiva del árido según la relación:
$Si_{(reactiva)} = Si_{(soluble\ total)} — 1{,}4\ Al_{(soluble\ total)}$

Por tanto, en áridos de similar porosidad y bajo contenido en aluminio, el valor de sílice soluble se podría correlacionar con la expansión generada por los áridos.

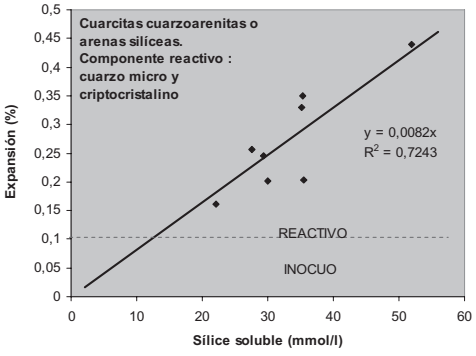
3.2 *Correlación sílice soluble—expansión*

A partir de áridos de reacción rápida por su contenido en cuarzo microcristalino y criptocristalino extraídos de obras afectadas por la reacción álcali sílice, se ha tratado de correlacionar la expansión en el ensayo acelerado de barras de mortero con la sílice soluble a 24 horas del ensayo químico ASTM.

Todos los áridos analizados son cuarcitas, cuarzoarenitas y arenas silíceas. Por tanto, su porosidad es similar al tener la misma naturaleza y su contenido en aluminio soluble muy bajo ya que no son rocas ricas en aluminosilicatos.

Se han analizado 8 áridos. En la gráfica 1 se muestra la correlación entre sílice soluble y la expansión en el ensayo acelerado de probetas de mortero.

Según la interpretación americana del ensayo acelerado de probetas de mortero, el árido es reactivo si la expansión es superior a 0,1% a los 14 días. Esta expansión corresponde a 12 mmol/l a las 24 horas.



Gráfica 1. Correlación entre ensayo acelerado de probetas de mortero y sílice soluble.

Si se considera que para que sea clasificado reactivo a 14 días la expansión debe ser superior al 0,2% (interpretación de la norma UNE) este límite coincide con un valor de 24 mmol/l de SiO_2 soluble a 24 horas. En este caso entre 12 y 24 mmol/l el árido es potencialmente reactivo y sería necesaria más información del árido.

Por tanto, aunque hay que tener en cuenta que el ensayo no es de aplicación a cualquier tipo de árido, sí se puede estimar de forma rápida si el árido será reactivo o inocuo cuando éstos son áridos de la misma naturaleza que los empleados para obtener la correlación anterior. Por tanto, resultados positivos clasifican en 24 horas al árido como reactivo, al igual que lo haría en ensayo acelerado de probetas de mortero a los 16 días.

4 DIFRACCIÓN DE RAYOS X

4.1 *Introducción (Rachmachandran 2001; Gispert i Guirado 2006)*

La difracción de rayos X es un fenómeno de interacción óptica entre los rayos X y los átomos de una estructura cristalina.

La difracción del polvo cristalino se realiza a una muestra con tamaño de partícula inferior a 0,06 mm formada por una o varias fases distintas, con la finalidad principal de identificar las fases que forman la muestra. A través de la difracción de rayos X se puede obtener una relación de valores de espaciados "d" y la intensidad relativa de cada reflexión respecto a las demás producidas, así como el ángulo en el que se ha producido cada una.

Existen bases de datos de difractogramas de más de 60.000 fases cristalográficas distintas (PDF Database, International Centre for Diffraction Data, ICDD). En el estudio de áridos, la comparación de los valores de "d", su intensidad

y su ángulo de reflexión obtenidos con la base de datos es suficiente para la identificación de los componentes mineralógicos del árido.

4.2 *Dominios cristalinos*

Al realizar la difracción en polvo de una muestra, las reflexiones teóricas serían reflexiones puntuales en unos ángulos de incidencia del haz de rayos X concretos (Figura 2). Sin embargo, las reflexiones son áreas aproximadamente triangulares centradas en la reflexión teórica, tal como se muestra en las Figura 3.

Las causas más importantes que generan el ensanchamiento de los picos difractados son (Pecharsky 2005; Gispert i Guirado 2006):

– Aberraciones geométricas introducidas por las características particulares del equipo de difracción de rayos X utilizado. Los parámetros más importantes que influyen en esta anchura de pico son: tipo de radiación empleada (Co, Mo, Fe, Cu, etc.), la tensión de la fuente (20–60 KV), corriente de filamento, la combinación de rendijas para la colimación y filtrado de la radiación, la alineación inicial del goniómetro, los ángulos de 2θ inicial y final del barrido, la resolución angular.

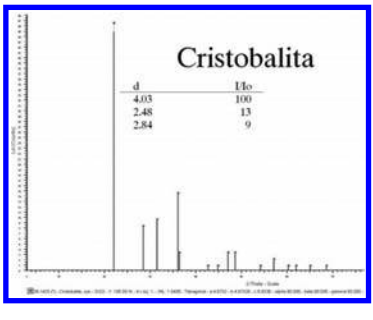


Figura 2. Difractograma teórico de la cristobalita.

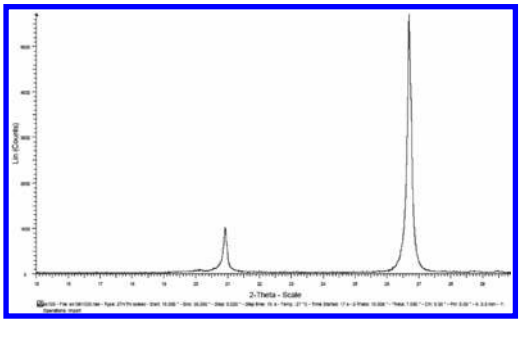


Figura 3. Difractograma real del sílex.

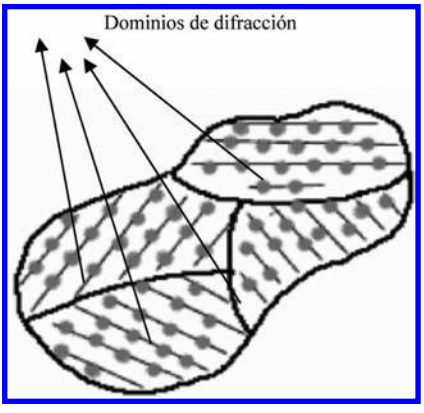


Figura 4. Dominios cristalinos.

– Tamaño de dominios cristalinos. Las deformaciones plásticas de un material dúctil producen ordenamientos y/o dislocaciones. Estas son responsables del fraccionamiento de los cristales originales dando dominios de difracción coherentes muy pequeños. Estos dominios de difracción están suficientemente desordenados unos respecto a sus próximos como para que entre ellos la difusión sea incoherente, lo cual implica, que la intensidad total difundida en un punto es simplemente la suma de las intensidades difractadas por cada dominio. Como los picos de difracción se encuentran muy próximos a los correspondientes a cada uno de los demás dominios vecinos, el diagrama de difracción presenta picos ensanchados.
– Imperfecciones en la red. Las dislocaciones que causan subdivisión de los cristales causan subdivisión en distintos cristalitos (distintos dominios de difracción). También las tensiones de tracción y compresión dentro de los mismos producen el ensanchamiento de las líneas.

El cálculo del tamaño medio de los dominios cristalinos se puede relacionar con la anchura media de las líneas de difracción de rayos X por medio de la ecuación de Scherrer:

$$T = k \cdot \lambda / B \cdot \cos\theta_B$$

– T: tamaño de cristal
– λ: longitud de onda de los rayos X.
– k: constante de forma del cristal (normalmente 0,9)
– B: la anchura a media altura de la linea de difracción
– θ_B: ángulo de difracción

4.3 *Detección de formas reactivas de cuarzo por difracción de rayos X*

En la aplicación de la difracción de rayos X a la RAS, hay que tener en cuenta que los componentes de sílice reactivos con los álcalis del hormigón son: tridimita, cristobalita, ópalo, vidrio volcánico, calcedonia y cuarzo altamente deformado o con tamaño de grano micro y criptocristalino (tamaño de grano menor de 60 µm se clasifica como reactivo). Así, en función de los resultados de difracción que se obtienen, los componentes reactivos se pueden subdividir en tres grupos:

- Ópalo Amorfo y Vidrio volcánico. Son amorfos, por lo que son sólo detectables por difracción cuando están presentes en un % elevado. Como ejemplo, el mínimo detectable de vidrio en cuarzo: 40% (Pawloski 1985).
- Tridimita, Cristobalita, Ópalo-C y Ópalo-CT. Estos componentes presentan estructura cristalina propia, por tanto presentan un difractograma característico.
- Calcedonia, cuarzo microcristalino, cuarzo criptocristalino y/o altamente deformado. Son formas diferentes del cuarzo por lo que presentan todas el mismo difractograma.

En este apartado de la comunicación se ha tratado de correlacionar la reactividad de los áridos por su contenido en cuarzo reactivo con el tamaño de dominios cristalinos (DS).

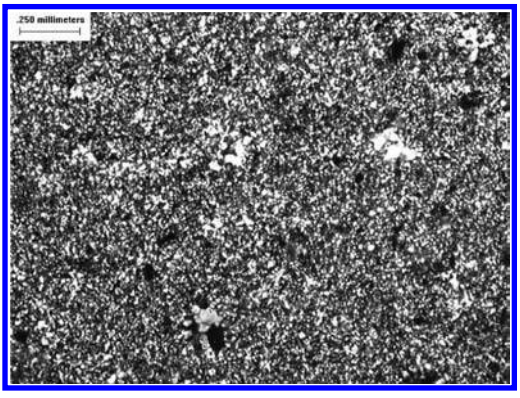
Tradicionalmente se ha empleado el índice de cristalinidad del cuarzo (QCI) evaluado a través de la difracción, si bien no ha dado buenos resultados (Broekmans 2004). Los dominios cristalinos se han empleado para comparar de forma cualitativa diferentes tipos de sílex, pero no aparece en la bibliografía ningún límite que correlacione claramente la reactividad en función del valor de dominios cristalinos.

Para obtener la correlación entre ambos parámetros se han realizado mezclas de sílex (cuarzo criptocristalino reactivo, fotografía 6) con una matriz de cristal de cuarzo inocuo (fotografía 7).

En la tabla 3 y gráfica 2 se muestran los resultados de la correlación entre el contenido en cuarzo criptocristalino reactivo y el tamaño de dominios cristalinos.

Se obtiene una buena correlación entre el tamaño de dominios cristalinos en los ensayos de laboratorio realizados para definir los límites de la técnica.

Si se toma como referencia el límite del 3% para el sílex que fija Portland Cement Association (Malvar 2002), se puede estimar que valor de dominios cristalinos diferencia áridos reactivos e inocuos. Con el modelo evaluado y el límite del 3%, los áridos serán reactivos cuando el valor de dominios cristalinos sea inferior a 1420 A.



Fotografía 6. Sílex. LPA.

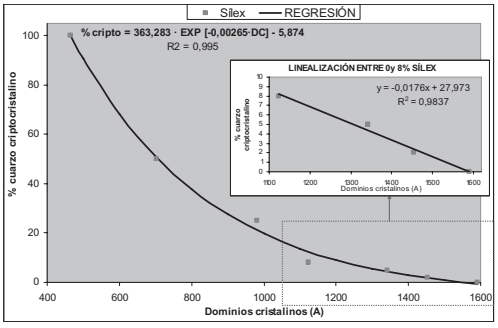
Fotografía 7. Cristal de cuarzo.

Tabla 3. Sílex en cuarzo inocuo.

% de sílex en matriz de cuarzo no reactivo	Tamaño dominios cristalinos (Å)
Matriz cuarzo	1591
2%	1454
5%	1342
8%	1124
25%	980
50%	702
Sílex	465

Este ensayo será válido para el estudio de áridos silíceos cuyo componente mayoritario sea el cuarzo.

Se puede evaluar si el contenido en cuarzo reactivo criptocristalino es suficiente para que el árido sea reactivo. También es una técnica útil cuando en áridos calizos aparece cuarzo como compuesto



Gráfica 2. Correlación entre dominios cristalinos y el contenido de cuarzo criptocristalino.

minoritario. Este cuarzo puede ser inocuo o reactivo pero su estudio petrográfico es complejo porque aparece diseminado en una matriz caliza. Por ataque ácido se puede extraer el cuarzo y estimar su reactividad por dominios cristalinos.

5 DISCUSIÓN DE RESULTADOS

En esta comunicación se han evaluado tres ensayos que de forma rápida (máximo tres días) proporcionan información sobre la reactividad de los áridos. A cambio, los ensayos tienen limitaciones que es necesario conocer para no obtener conclusiones erróneas de sus resultados.

A continuación se expone la aplicación y limitaciones de cada uno de ellos.

5.1 *Gel Pat modificado*

En el Laboratorio Central de Estructuras y Materiales se plantea un nuevo ensayo a partir del ensayo Gel Pat original variando la granulometría de la muestra y la forma de fabricar las pastillas para así favorecer la formación de geles y la extracción de partículas. También se ha aumentado la temperatura, así se acelera la reacción y la disminución de la viscosidad de éstos facilita que se depositen en huecos y fisuras.

Este ensayo es válido para detectar áridos reactivos, tanto rápidos como lentos, y sobre todo clasificar de forma rápida los áridos inocuos. Permite en tres días saber si el árido es potencialmente reactivo e identificar las partículas reactivas. De esta forma, complementando con un número pequeño de láminas delgadas, se puede conocer el tipo de componente reactivo y aplicar en cada caso, el ensayo más adecuado para evaluar el grado de reactividad del árido.

La ejecución del ensayo es sencilla, lo que permite, cuando se disponen de distintas canteras para la ejecución de una obra hidráulica, conocer que áridos son potencialmente reactivos y cuales son inocuos.

El inconveniente del ensayo es que la valoración de la reactividad es cualitativa. A cambio, discrimina rápidamente áridos inocuos. Si el ensayo se clasifica como potencialmente reactivo es necesario disponer del ensayo más adecuado en cada caso para evaluar el grado de reactividad del árido.

Es necesario ampliar la muestra de ensayo con más áridos reactivos reales con diferentes tipos de componente reactivo para comprobar si el ensayo es de aplicación a todos los componentes reactivos posibles.

5.2 *Ensayo químico*

La medida de la sílice soluble de la fracción 0,150–0,300 mm siguiendo la norma ASTM se ha correlacionado con la expansión en el ensayo acelerado de barras de mortero a 14 días. De esta forma en 24 horas y evaluando un solo parámetro se puede estimar si el árido será potencialmente reactivo o inocuo.

Existen otros parámetros que también afectan a la expansión como son el contenido en aluminosilicatos del árido o su porosidad. El ensayo se ha realizado para áridos silíceos (cuarcitas, cuarzoarenitas y arenas silíceas) en los que la porosidad no ha influido en la relación expansion—sílice soluble y con bajo contenido en aluminio. Por tanto, la correlación está restringida a este tipo de áridos.

Disoluciones superiores a 12 mmol/l superan la expansión de 0,1% a 14 días, por lo que clasificarían al árido como reactivo (interpretación más restrictiva del ensayo acelerado de barras de mortero). Esta correlación sería válida para áridos de reacción rápida por su contenido en cuarzo micro y criptocristalino. En el caso de cuarzo microfisurado y deformado, el árido es de reacción lenta y estos límites no serían válidos, por lo que un resultado negativo en este ensayo podría significar que el árido es de reacción lenta.

Es necesario ampliar la muestra de estudio, principalmente en la zona que limita los áridos reactivos e inocuos y la zona de áridos inocuos para comprobar que la relación lineal se mantiene en todo el rango de estudio y evaluar la sensibilidad del método para clasificar correctamente a los áridos inocuos.

5.3 *Difracción de rayos X*

Se ha empleado la difracción de rayos X para tratar de diferenciar formas de cuarzo reactivas e inocuas.

El tamaño de dominios cristalinos muestra una buena correlación con el contenido en sílex de la muestra y, por tanto, con su reactividad.

La técnica es válida cuando se quiere saber en áridos cuyo componente mineralógico principal es el cuarzo, si éste será reactivo por su contenido en formas reactivas de cuarzo.

Según el modelo definido, un valor de dominios cristalinos inferior a 1420 A equivaldría a un contenido en sílex del 3%, por lo que valores inferiores o iguales a éste clasificarían al árido como reactivo.

El límite para el tamaño de dominios cristalinos debe ser corroborado con áridos reales de los que sea conocida su reactividad y con formas de cuarzo reactivas diferentes de sílex.

6 CONCLUSIONES

Gel-Pat modificado

- El ensayo Gel Pat modificado es capaz, en tres días, de identificar áridos potencialmente reactivos por componentes de reacción rápida y lenta.
- Se pueden identificar partículas reactivas y extraerlas de forma sencilla para su estudio petrográfico e identificación de componentes reactivos.
- El ensayo es cualitativo. Si se clasifica el árido como potencialmente reactivo es necesario emplear, en cada caso, el ensayo más adecuado para evaluar el grado de reactividad del árido. Si el ensayo es negativo el árido es inocuo (rápido o lento).

Sílice soluble

- La sílice soluble evaluada según la norma ASTM de reactividad álcali sílice se ha correlacionado con la expansión en el ensayo acelerado de probetas de mortero.
- La correlación es válida para áridos silíceos con bajo contenido en aluminosilicatos (cuarcitas, cuarzoarenitas y arenas siliceas).
- Una disolución de 12 mmol/l para este tipo de áridos corresponde con una expansión de 0,1% en el ensayo acelerado de barras de mortero a 14 días.
- Si el ensayo da positivo el árido es reactivo sólo para áridos rápidos. Si el ensayo es negativo, podría ser reactivo por componentes reactivos de reacción lenta (cuarzo microfracturado o deformado).

Difracción de rayos X

- La medida de dominios cristalinos del cuarzo por difracción de rayos X ha mostrado buena correlación con el contenido en sílex en el estudio de laboratorio realizado.
- El contenido máximo del 3% en sílex coincide con un valor de dominios cristalinos de 1420 A. Por debajo de este valor los áridos se clasificarían como reactivos por su contenido en cuarzo criptocristalino.

AGRADECIMIENTOS

Este trabajo ha sido realizado por el CEDEX enmarcado dentro del convenio: "Estudio del deterioro de los hormigones de presas" de la Dirección General del Agua (Ministerio de Medio Ambiente y Medio Rural y Marino).

Esta investigación también es parte de los trabajos desarrollados por el CEDEX en el proyecto del Plan Nacional de I+D (2004–2007) BIA-09412-C03-02: Desarrollo de un sistema integrado para la medida y gestión de estructuras de hormigón con problemas expansivos.

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Relación geología-patología en presas españolas afectadas por la reacción álcali-sílice

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RESUMEN: En la reacción álcali-sílice (RAS) juega un papel fundamental la presencia de sílice inestable que puede contener algunos áridos empleados en el hormigón (cuarcitas, granitos, etc.). Sin embargo, la selección de estos áridos debe realizarse entre las rocas del entorno geológico próximo a su ubicación, para evitar largas distancias de transporte. Este condicionante restringe de forma importante los áridos a emplear, debiéndose, tras un estudio pormenorizado, seleccionar el más adecuado entre los cercanos a la obra.

Los Mapas Geológicos de España ofrecen una información extremadamente útil para detectar la posible presencia de áridos que pueden desencadenar una reacción álcali-sílice. El análisis realizado del estudio petrográfico de los áridos empleados en diferentes obras muestra la estrecha relación existente entre la información geológica disponible en los mapas consultados y el riesgo de aparición de una patología por reacción álcali-sílice, pudiendo establecer así un "Mapa de riesgo de patología geológica por reacción álcali-sílice".

ABSTRACT: The presence of metaestable silica in some aggregates used in concrete (quartzites, granites, etc) plays an important role in the alkali-silica reaction (ASR). The aggregates are always selected among the rocks near the construction emplacement to avoid a long transport distance, which increases the cost.

The geologic maps of Spain provide extremely useful information to detect the presence of reactive aggregates, which could develop an alkali-silica reaction. The petrographic study carried out in aggregates used in different real works shows a close relationship between the geologic information in the maps and the risk to develop ASR pathology, so a "Risk of ASR pathology map" can be established.

1 INTRODUCCIÓN

En la reacción álcali-sílice (RAS), la sílice de ciertos áridos es atacada por los iones OH^- de la solución de los poros del hormigón y es liberada, originándose una reacción química con la formación de un gel, que al embeber en agua, aumenta de volumen. Las presiones generadas por el aumento de volumen producen el deterioro del hormigón.

Según sea la naturaleza de los áridos se distinguen dos tipos diferentes de reacción, dependiendo de los componentes que presenten los áridos y de la velocidad con la que éstos reaccionen en el seno del hormigón, presentándose así, una reacción rápida y una reacción lenta (Hobbs 1988) (Poole 1992) (ACI 1998) (Ponce and Batic 2006). Un primer grupo de rocas es el compuesto por aquéllas que contienen especies amorfas, especies porosas o minerales hidratados como el ópalo, que son extremadamente reactivos en determinadas condiciones (Wigum and French 1996; Ponce and Batic 2006). El otro grupo está formado aquellas rocas silíceas bien cristalizadas y más densas que, bajo deformación o cambios térmicos, pueden presentar cuarzos deformados y/o microfisurados que muestran ciertos niveles de reactividad mediante un mecanismo lento que produce una reacción con efectos expansivos y un deterioro tardío.

Varias obras hidráulicas y algún puente han manifestado este tipo de patología en España, existiendo constancia de 16 obras ya diagnosticadas, que se representan marcadas y numeradas en el mapa de la Figura 1. Los áridos empleados en la fabricación del hormigón en estas obras eran tanto rocas graníticas como rocas no graníticas, siendo la aparición de esta patología más frecuente en el caso de las primeras.

En el mapa de la figura anterior se diferencian las obras afectadas por RAS lenta (números del 1 al 11) asociadas a la presencia de rocas graníticas y las afectadas por RAS rápida (números del 12 al 16) asociadas a las rocas metamórficas

Rocas		Minerales sensibles en medio alcalino
Magmáticas	– Granitos – Granodioritas	Cuarzo con red deformada presentando una extinción ondulante. Minerales feldespáticos alterados, uniones de partículas abiertas.
	– Riolitas – Dacitas – Andesitas – Traconandesitas – Basaltos	Presencia de vidrios silíceos o vidrios basálticos más o menos desvitrificados, presencia de tridimita, cristobalita y ópalo.
	– Obsidiana – Tufos volcánicos – Retinitos	Vidrios ricos en sílice más o menos desvitrificados y frecuentemente figurados.
Metamórficas	– Gneis – Mica-esquistos	Cuarzo con extinción ondulatoria. Microcuarzo de segunda generación, uniones de partículas abiertas, minerales feldespáticos y micáceos alterados.
	– Cuarcitas – Corneanas	Cuarzos asociados a un cemento cuarzoso y opalino. Presencia de microcuarzo de segunda generación. Presencia de filosilicatos. Presencia de cuarzo con extinción ondulatoria o de cuarzo microfisurado.
Sedimentarias	– Gres – Cuarcitas	Cemento silíceo mal cristalizado, uniones de partículas alargadas.
	– Grauwacas – Sititas – Esquistos cuarzosos	Minerales filíticos asociados. Presencia de ópalo de cuarzo microcristalizado.
	– Silex	Presencia de calcedonia, ópalo.
	– Calizas – Calizas dolomíticas – Dolomitas	Presencia de sílice de tipo ópalo en microdominios o distribuidas en red, asociada o no con sulfuros sedimentarios y filitas.

Tabla 1. Listado de las rocas reactivas frente a la reacción álcali-sílice según EHE-08.

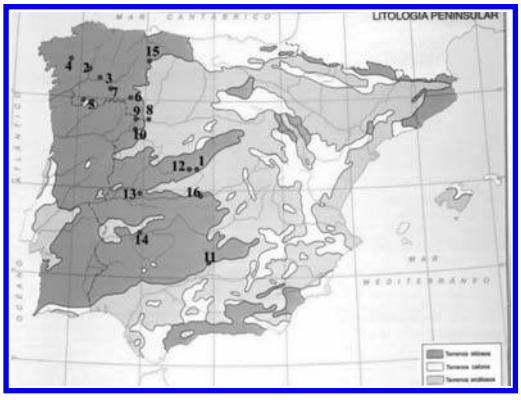

Figura 1. Mapa geológico con la ubicación de las obras afectadas.

(cuarcitas, esquistos y gneises) y sedimentarias (cuarzoarenitas, sílex, etc. …).

La Instrucción de hormigón EHE-08 incluye una serie de rocas susceptibles de desarrollar la reacción álcali-sílice y presenta en la tabla 1 una relación de los nombres geológicos de las rocas y sus potenciales componentes reactivos. Esta tabla está tomada de la experiencia de otros países, incluyendo un amplio listado de rocas que han dado lugar a problemas.

La geología de la península Ibérica se divide tradicionalmente en tres grandes grupos, que se muestran en la Figura 1: terrenos silíceos, calizos y arcillosos, siendo el grupo de la España silícea el más importante para este estudio, estando compuesta de los materiales más antiguos y ricos en sílice.

Dentro de esta zona silícea podremos encontrar rocas plutónicas (como las rocas graníticas) y metamórficas (como pizarras, esquistos y cuarcitas).

En la zona detrítica española (terrenos arcillosos), desde un punto de vista teórico, las rocas presentes podrían dar también la reacción álcali-sílice, por ejemplo las cuencas de los grandes ríos, aunque en la práctica no se han producido casos, concentrándose todos en la España estrictamente designada como silícea.

2 OBJETIVOS Y METODOLOGÍA

El objetivo propuesto en el estudio que se presenta en esta ponencia ha sido analizar los diferentes macizos rocosos de naturaleza silícea que han dado problemas de reactividad en España y el componente reactivo encontrado en los mismos. Los resultados se han comparado con las tipologías de rocas reactivas recogidas en la Instrucción EHE, para así conocer específicamente los tipos de rocas problemáticas en nuestro país y simplificar el largo listado que recoge la Instrucción.

Para ello se ha procedido a un estudio exhaustivo de los mapas geológicos de las zonas dónde se sitúan las obras afectadas por la reacción álcali-sílice y se ha extraído la información sobre los materiales pétreos recogidos en estos mapas y en sus memorias, específicamente aquellos datos que tienen una influencia directa en su empleo como áridos para hormigón.

Por otro lado, se ha realizado un examen petrográfico de las láminas delgadas de los áridos empleados en la fabricación del hormigón de las obras afectadas.

3 OBRAS AFECTADAS POR LA RAS LENTA

Se han analizado las memorias de los mapas geológicos 1:50.000 de España, publicadas por el IGME (instituto Geológico y Minero de España), en especial las descripciones petrográficas de los materiales que en ellos se encuentran.

Para la selección de los mapas geológicos se ha tenido en cuenta las zonas donde existen evidencias de reacción álcali-sílice en obras reales y los materiales con los que el hormigón de éstas estaba fabricado.

3.1 *Obra 1*

De acuerdo con el mapa geológico en el entorno de la obra se localizan dos macizos con indicios de reactividad, que se resaltan en el mapa de la Figura 2. Éstos son las adamellitas de Las Navas del Marqués-San Martín de Valdeiglesias y los leucogranitos.

La lectura del resumen del mapa geológico alerta de la presencia en las rocas del entorno de la obra de cuarzo con extinción ondulante, y fracturado, que es un componente reactivo lento. De estos macizos se extrajo el árido para la fabricación del hormigón, afectado por RAS lenta.

El análisis petrográfico que se ha llevado a cabo en el estudio de los áridos con los que se fabricó el hormigón de esta obra para diagnosticar su patología, han clasificado las rocas como granodioritas y monzogranitos. Este estudio confirma la presencia de cuarzos con extinción ondulante, visible con luz polarizada analizada (LPA) y con cristales fracturados, coincidiendo con lo extraído de la memoria del mapa geológico y confirmando la reactividad de los materiales así resaltados en la Figura 3.

En concreto, los materiales empleados en la fabricación del hormigón se han podido identificar como adamellitas de Las Navas del Marqués-San Martín de Valdeiglesias, macizo cuya descripción geológica ya indicaba su reactividad potencial.

Figura 2. Mapa geológico de la zona de la obra.

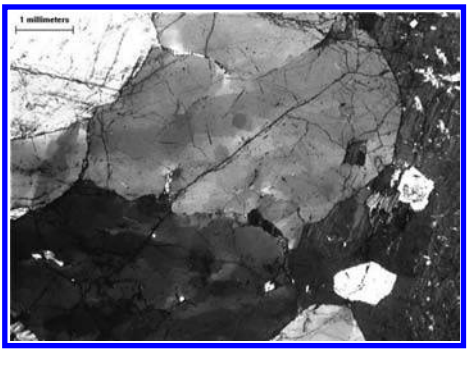

Figura 3. Cuarzo con extinción ondulante. LPA.

3.2 *Obra 2*

De acuerdo con el mapa geológico en el entorno de la obra se localizan cinco macizos con indicios de reactividad: Cuarcitas y areniscas ordovícicas, cuarcita Armoricana, granodiorita precoz con megacristales, granodiorita tardía con megacristales y granito alcalino de dos micas.

En estos materiales, los mapas geológicos consultados muestran que el cuarzo aparece con extinción ondulante, cristales orientados con los contactos suturados e interpenetrados, en algún caso el cuarzo aparece cataclástico y fuertemente recristalizado, características todas asociadas a la reacción álcali-sílice de tipo lento.

El análisis petrográfico que se ha llevado a cabo en el estudio de los áridos con los que se fabricó el hormigón de esta obra para diagnosticar su patología, ha clasificado las rocas como granodiorita. Este estudio confirma la presencia de cuarzos deformados con extinción ondulante y una leve recristalización (Figura 4), y relaciona los materiales empleados en la fabricación del hormigón con la granodiorita precoz con megacristales, que ya la memoria del mapa geológico señalaba como potencialmente reactivos.

Este macizo se resalta en el mapa de la Figura 5 como reactivo mientras que el resto de macizos, presenta características potencialmente reactivas pero sin confirmar, al no haberse utilizado en la fabricación del hormigón, y por ello se marcan como potencialmente reactivos en la figura 5 de acuerdo con su descripción en el mapa geológico.

3.3 *Obra 3*

De acuerdo con el mapa geológico en el entorno de la obra se localizan seis macizos con indicios de reactividad: Granodiorita precoz, granito orientado

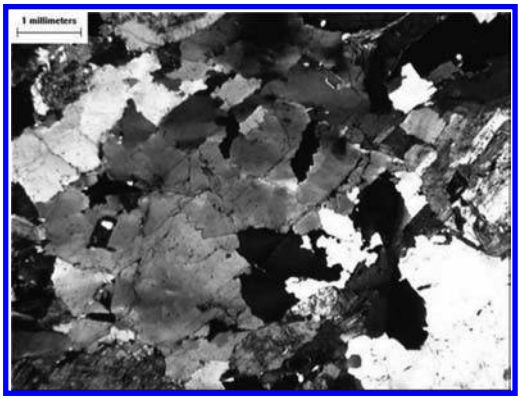

Figura 4. Cuarzo deformado con extinción ondulante y recristalización. LPA.

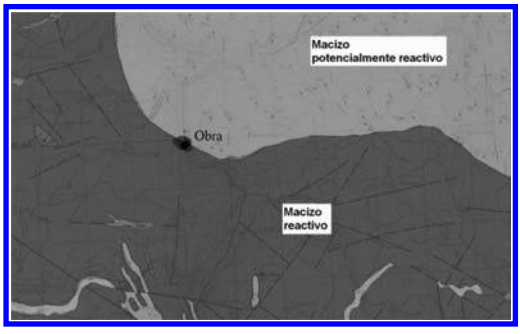

Figura 5. Mapa geológico de la zona de la obra.

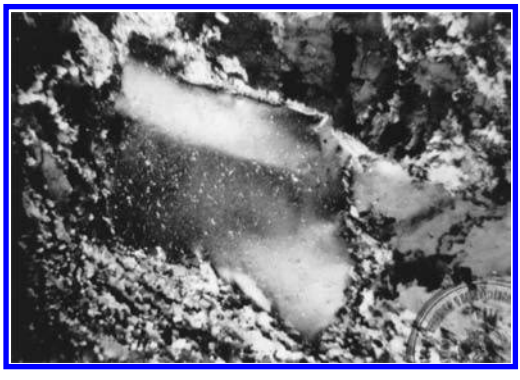

Figura 6. Cuarzo con extinción ondulante y recristalización. LPA.

de dos micas, granito adamellítico, granito de dos micas porfiroide, granodiorita tardía y cuarcitas.

En todos los macizos graníticos, el mapa geológico señala que el cuarzo se presenta como cristales alotriomorfos, con extinción ondulante, con mayor o menor intensidad, y textura en mortero en algún caso, lo que es signo de una potencial reactividad álcali-sílice de tipo lento. En el caso particular de las cuarcitas, los granos de cuarzo se presentan orientados, lo cual puede dar una idea de una cierta deformación de los cristales de cuarzo ocasionado por metamorfismo y en tal caso a una potencial RAS lenta.

El análisis petrográfico que se ha llevado a cabo en el estudio de los áridos con los que se fabricó el hormigón de esta obra para diagnosticar su patología, han clasificado las rocas como granitos y granodioritas. Este estudio confirma la presencia de cuarzos con cristales muy deformados y fracturados, con extinción ondulante y cuarzo sacaroideo (textura en mortero) recristalizado y cuarzo de grano fino (Figura 6) y relaciona los materiales empleados en la fabricación del hormigón con el granito de dos micas orientado y la granodiorita

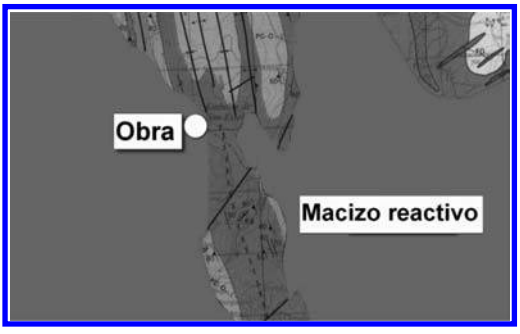

Figura 7. Mapa de la zona de la obra.

precoz, que ya la memoria del mapa geológico señalaba como potencialmente reactivos.

Así, estos macizos se resaltan en el mapa de la Figura 7 como reactivos de acuerdo con su descripción en el mapa geológico.

3.4 *Obra 4*

De acuerdo con el mapa geológico en el entorno de la obra se localizan diez macizos con indicios de reactividad: Cuarzoesquistos y metagrauvacas (series de Ordenes), gneises migmatíticos, ortogneis pegmatoide, Ortogneis de Sobrado, Ortogneis de Merza, facies de ollo de sapo, granitos calcoalcalinos biotíticos, granito biotítico porfídico, granito de dos micas de grano medio a fino, leucogranito y diques de cuarzo.

Las características descritas en los cuarzos de los macizos anteriores indican la presencia de cuarzo de grano fino, con recrecimientos, con fracturación en los cristales y recristalizaciones en mortero, indicativas de que estos materiales podrían ser potencialmente reactivos para la RAS de tipo lento.

Dentro de las rocas ígneas y en los gneises se describe el cuarzo como alotriomorfo con marcada extinción ondulante y a veces milonitizado, lo que es característica de componentes potencialmente reactivos frente a RAS lenta.

En los materiales cuaternarios, representados por depósitos aluviales, no se encuentra información en el mapa geológico pero, al estar formados por cuarcitas y rocas del entorno podrían ser igualmente reactivas frente a la RAS de tipo lento.

El análisis petrográfico que se ha llevado a cabo en el estudio de los áridos con los que se fabricó el hormigón de esta obra para diagnosticar su patología, ha puesto de manifiesto que los áridos empleados para la fabricación del hormigón de esta obra se extrajeron de los depósitos aluviales. Estos depósitos están compuestos por varios tipos de rocas del entorno, siendo aquéllas pertenecientes

a los cuarzoesquistos, metagrauvacas y diques de cuarzo y de amplia representación en el mapa geológico de la zona, las detectadas y todas ellas potencialmente reactivas.

3.5 *Obra 5*

De acuerdo con el mapa geológico en el entorno de la obra se localizan cuatro macizos con indicios de reactividad: Nebulitas, granitos parautóctonos, granito de dos micas y granodioritas biotíticas.

La lectura del resumen del mapa geológico alerta de la presencia en las rocas del entorno de la obra con cuarzo con extinción ondulante, recristalizado y microfisurado y con agregados en mosaico, características todas asociadas a la reacción álcali-sílice de tipo lento. De estos macizos se extrajo el árido para la fabricación del hormigón, afectado por RAS lenta.

El análisis petrográfico que se ha llevado a cabo en el estudio de los áridos con los que se fabricó el hormigón de esta obra para diagnosticar su patología, han clasificado las rocas como graníticas. Este estudio confirma la presencia de cuarzos deformados con extinción ondulante y relaciona los materiales empleados en la fabricación del hormigón con el granito de dos micas o granodioritas biotíticas, que ya la memoria del mapa geológico señalaba como potencialmente reactivos, y así aparecen señalados como reactivos en la Figura 8.

3.6 *Obra 6*

De acuerdo con el mapa geológico en el entorno de la obra se localizan seis macizos con indicios de reactividad: Gneises de la formación "ollo de sapo", formación Puebla, formación culebra, migmatitas con núcleos de Ribadelago, granitos biotíticos a granodioritas y granito de dos micas de Calabor.

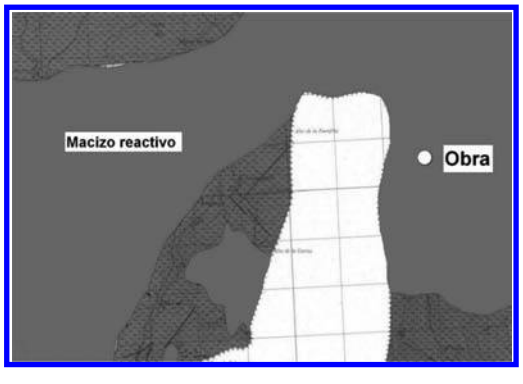

Figura 8. Mapa geológico de la zona de la obra.

Figura 9. Mapa geológico de la zona de la obra.

La lectura del resumen del mapa geológico alerta de la presencia en estas rocas de cuarzo con extinción ondulante, bordes suturados y recristalización que son características de áridos de reacción lenta. En algunos casos estas características pueden estar más acentuadas, como en las migmatitas. De estos macizos se extrajo el árido para la fabricación del hormigón, afectado por RAS lenta.

El análisis petrográfico que se ha llevado a cabo en el estudio de los áridos con los que se fabricó el hormigón de esta obra para diagnosticar su patología, ha clasificado las rocas como granodioritas. Este estudio confirma la presencia de cuarzos deformados con extinción ondulante y relaciona los materiales empleados en la fabricación del hormigón con los granitos biotíticos y granodioritas, que ya la memoria del mapa geológico señalaba como potencialmente reactivos y se resaltan como reactivos en la Figura 9.

Aunque en la obra no se haya producido deterioro externo, en un estudio detallado de los áridos extraídos de la obra se ha podido comprobar que sí se producía reacción álcali-sílice de lo cual puede deducirse que el alcance de la misma es limitado.

3.7 *Obra 7*

De acuerdo con el mapa geológico en el entorno de la obra se localizan nueve macizos con indicios de reactividad: Ortogneis, granito de dos micas, granodioritas tardías, granodioritas precoces, granitos de grano medio, granito de dos micas de grano medio a grueso, cuarcitas de la serie de Viana, liditas y granito de dos micas deformado de Calabor.

La lectura del resumen del mapa geológico alerta de la presencia en las rocas del entorno de la obra de cuarzos fracturados, deformados con extinción ondulante, recristalizaciones y textura en mortero, características todas ellas de la reacción álcali-sílice de tipo lento y puntualmente con un tamaño de grano criptocristalino, que podría indicar superposición de una RAS rápida. De estos macizos se extrajo el árido para la fabricación del hormigón, afectado por RAS.

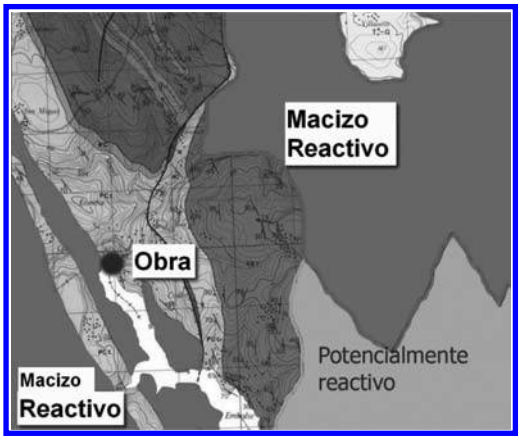

Figura 10. Mapa geológico de la zona de la obra.

El análisis petrográfico que se ha llevado a cabo en el estudio de los áridos con los que se fabricó el hormigón de esta obra para diagnosticar su patología, ha clasificado las rocas como granodioritas. Este estudio confirma la presencia de cuarzos deformados con extinción ondulante y relaciona los materiales empleados en la fabricación del hormigón con los granitos de dos micas, granodioritas tardías y granodioritas precoces, que ya la memoria del mapa geo-lógico señalaba como potencialmente reactivos, y resaltados como reactivos en la Figura 10.

Sin embargo, las rocas marcadas como potencialmente reactivas en la misma figura se corresponden con zonas en las que la memoria informa sobre la presencia de componentes potencialmente reactivos pero que no se tiene la certeza de su reactividad al no haberse utilizado en la fabricación del hormigón.

Hay que señalar que en esta obra no se ha producido deterioro, aunque un estudio detallado de testigos extraídos de la misma se ha podido comprobar que sí se había producido una reacción álcali-sílice, aunque de forma leve sin sintomatología aparente.

3.8 *Obras 8 y 9*

De acuerdo con el mapa geológico en el entorno de la obra se localiza un macizo con indicios de reactividad: Granodiorita precoz.

La lectura del resumen del mapa geológico alerta de la presencia en este macizo de cuarzo, deformado con extinción ondulante y textura en mortero, características todas ellas de la reacción álcali-sílice de tipo lento. De estos macizos se extrajo el árido para la fabricación del hormigón, afectado por RAS lenta.

Aunque en la obra 8 no se haya producido deterioro, en un estudio detallado de testigos extraídos de la misma se ha podido comprobar que sí se producía reacción álcali-sílice, aunque de forma leve sin sintomatología aparente.

La obra 9 presenta un marcado deterioro producido por áridos rodados de cuarcita con extinción ondulante, sin existir una descripción petrográfica en la hoja del mapa geológico correspondiente.

3.9 *Obra 10*

De acuerdo con los mapas geológicos en el entorno de la obra se localizan dieciséis macizos con indicios de reactividad entre los que se destacan, por estar en las proximidades de la obra: Granito de dos micas porfiroide, granito de dos micas con textura "ala de mosca" y granito de grano medio de dos micas tipo Villar de Peralonso. La lectura del resumen del mapa geológico alerta de la presencia en las rocas del entorno de la obra de cuarzos deformados con extinción ondulante y con tendencia a formar subgranos y poligonización, en algún caso con bordes indentados deformados que son características de áridos de reacción lenta. De estos macizos se extrajo el árido para la fabricación del hormigón, afectado por RAS lenta.

El análisis petrográfico que se ha llevado a cabo en el estudio de los áridos con los que se fabricó el hormigón de esta obra para diagnosticar su patología, han clasificado las rocas como granitos y granodioritas. Este estudio confirma la presencia de cuarzos deformados con extinción ondulante y relaciona los materiales empleados en la fabricación del hormigón con el granito de dos micas con textura "ala de mosca", que ya la memoria del mapa geológico señalaba como potencialmente reactivos.

3.10 *Obra 11*

De acuerdo con el mapa geológico en el entorno de la obra se localizan dos macizos con indicios de reactividad: Granitos y granitos porfídicos.

La lectura del resumen del mapa geológico alerta de la presencia en las rocas del entorno de la obra de cuarzos deformados con extinción ondulante, fracturación y recristalización, que son características de áridos de reacción lenta. De estos macizos se extrajo el árido para la fabricación del hormigón, afectado por RAS lenta.

El análisis petrográfico que se ha llevado a cabo en el estudio de los áridos con los que se fabricó el hormigón de esta obra para diagnosticar su patología, han clasificado las rocas como granitos. Este estudio confirma la presencia de cuarzos deformados con extinción ondulante que ya la memoria del mapa geológico señalaba como potencialmente reactivos.

4 OBRAS AFECTADAS POR LA RAS RÁPIDA

4.1 *Obra 12*

De acuerdo con el mapa geológico en el entorno de la obra se localizan tres macizos con indicios de reactividad: Adamellitas-granodioritas foliadas porfídicas, granitos foliados y adamellitas con agregados biotíticos.

La lectura del resumen del mapa geológico alerta de la presencia en las rocas del entorno de la obra de cuarzo deformado con extinción ondulante (componente de la RAS lenta) y de cuarzo microcristalino que, en determinadas proporciones origina una RAS rápida. De estos macizos se extrajo el árido para la fabricación del hormigón, afectado por RAS rápida.

El análisis petrográfico que se ha llevado a cabo en el estudio de los áridos con los que se fabricó el hormigón de esta obra para diagnosticar su patología, han clasificado las rocas como granodioritas. Este estudio confirma la presencia de cuarzos muy deformados con extinción ondulante, con recristalización y cuarzo microcristalino en una proporción reactiva para el hormigón (Figura 11), causante de una RAS rápida. Así mismo, se relacionan los materiales empleados en la fabricación del hormigón con los indicados por el mapa geológico como adamellitas foliadas porfídicas, que ya la memoria del mapa geo-lógico señalaba como potencialmente reactivos, y resaltados en la Figura 12.

4.2 *Obra 13*

De acuerdo con el mapa geológico en el entorno de la obra se localizan ocho macizos con indicios de reactividad: 4 macizos metamórficos (cuarcita Armoricana, cuarcitas ordovícicas, cuarcitas intercaladas entre las pizarras, cuarcitas silúricas), facies sedimentarias terciarias en forma de sedimentos aluviales y 3 macizos ígneos (leucogranito, granito de dos micas y diques de cuarzo).

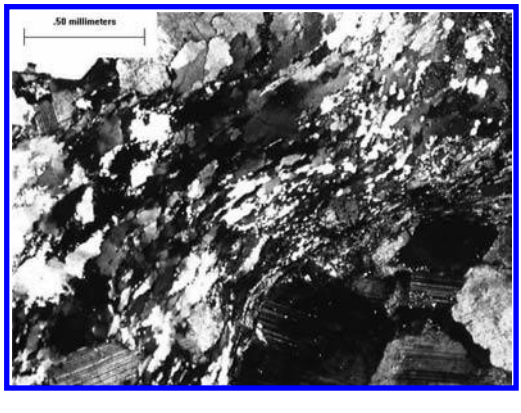

Figura 11. Cuarzo muy deformado y de tamaño microcristalino.

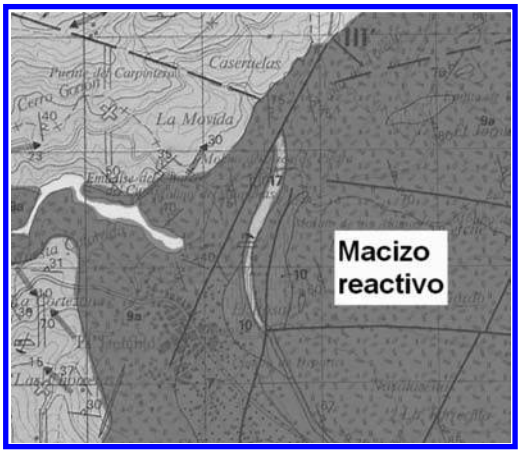

Figura 12. Mapa geológico de la zona de la obra.

Los materiales metamórficos están compuestos principalmente por cuarcitas y pizarras, donde el cuarzo aparece con tamaños que rondan las 20 µm, o cristales deformados, recristalizados con contactos suturados. También se ha encontrado cemento silíceo y cemento compuesto por calcedonia. Todos estos componentes están asociados a RAS rápida. Los materiales ígneos están representados por rocas plutónicas con cristales de cuarzo deformados con extinción ondulante con contactos suturados y poligonización, características de RAS lenta.

El análisis petrográfico que se ha llevado a cabo en el estudio de los áridos con los que se fabricó el hormigón de esta obra para diagnosticar su patología, han clasificado las rocas como cuarcitas. Este estudio confirma la presencia de cuarzos muy deformados con contactos suturados y cuarzo micro (entre 60 y 20 µm) y criptocristalino (menor de 20 µm), características de áridos de reacción rápida (Figura 13).

Puesto que los áridos de la obra están extraídos de los terrenos aluviales cercanos es lógico pensar que estos terrenos están formados por la erosión y transporte de los materiales cercanos, con lo que se pueden relacionar los áridos extraídos del hormigón con los niveles cuarcíticos cercanos. Éstos son la cuarcita Armoricana y las cuarcitas silúricas y ordovícicas, coincidiendo así con la memoria del mapa geológico que ya clasificaba estos macizos como potencialmente reactivos y se resaltan en el mapa de la Figura 14.

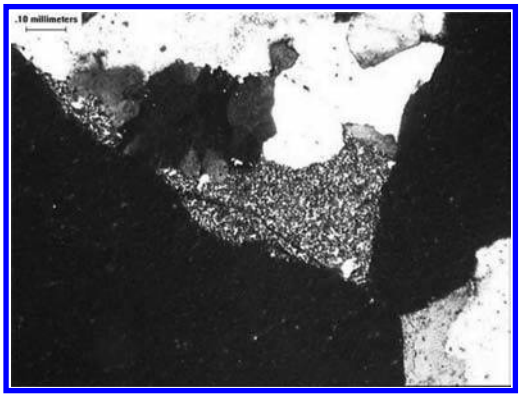

Figura 13. Cuarzo micro-criptocristalino. LPA.

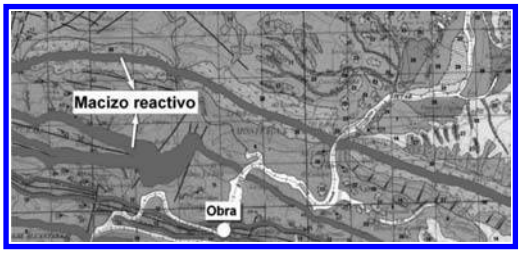

Figura 14. Mapa geológico de la zona de la obra.

4.3 *Obra 14*

De acuerdo con el mapa geológico en el entorno de la obra se localizan cuatro macizos con indicios de reactividad: Arenisca y cuarcita del Tremadociense, cuarcita Armoricana, cuarcita blanca masiva y cuarcitas ferruginosas.

La lectura del resumen del mapa geológico alerta de la presencia en las rocas cuarcíticas del entorno de la obra de cristales de cuarzo deformados con extinción ondulante y bordes serrados, con recristalizaciones en los bordes de grano, características todas de la RAS rápida.

El análisis petrográfico que se ha llevado a cabo en el estudio de los áridos con los que se fabricó el hormigón de esta obra para diagnosticar su patología, han clasificado las rocas como cuarcitas (Figura 14). Este estudio confirma la presencia de cuarzo deformado (RAS lenta) con cemento silíceo y cuarzo micro (entre 60 y 20 µm) y criptocristalino (menor de 20 µm), características de áridos de reacción rápida, en coincidencia con lo comentado en el mapa geológico.

4.4 *Obra 15*

Para estas obras la información de los mapas geológicos es muy restringida, centrándose únicamente en la información estratigráfica y sedimentaria y dejando de lado la caracterización petrográfica.

El análisis petrográfico que se ha llevado a cabo en el estudio de los áridos con los que se fabricó el hormigón de esta obra para diagnosticar su patología, han clasificado las rocas como cuarcitas y cuarzoarenitas con cemento silíceo criptocristalino. Estas rocas también presentan vénulas de cristales de cuarzo de tamaño micro y criptocristalino (Figura 15). Todas estas características pertenecen a áridos de reacción rápida.

4.5 *Obra 16*

Al igual que la obra anterior el mapa geológico no recoge la información petrográfica de los materiales encontrados en el entorno de la obra.

El análisis petrográfico que se ha llevado a cabo en el estudio de los áridos con los que se fabricó el hormigón de esta obra para diagnosticar su

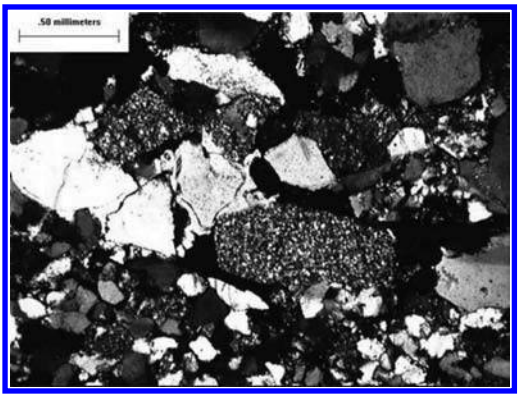

Figura 15. Cuarcita con cemento silíceo.

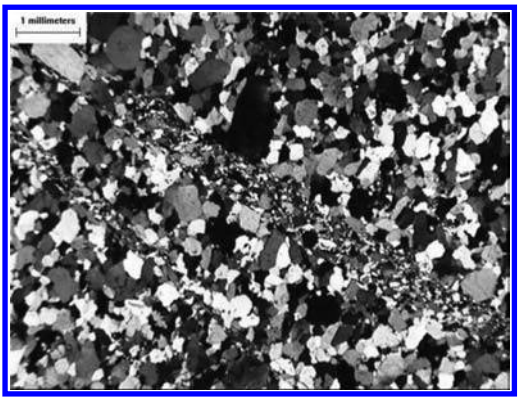

Figura 16. Vénula de cuarzo micro y criptocristalino en cuarcita.

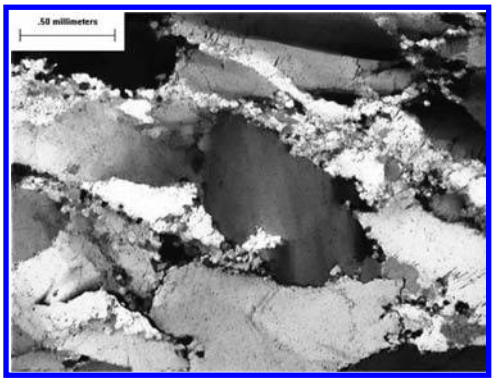

Figura 17. Cuarzos deformados con recristalización y cuarzo microcristalino en los bordes de grano.

patología, han clasificado las rocas como cuarcitas y cuarzoarenitas (figura 15) donde los cristales de cuarzo presentan una deformación intensa con bordes de grano suturados y recristalización. Se ha encontrado también cuarzo micro y criptocristalino. Todas estas características pertenecen a áridos de reacción rápida.

5 ANÁLISIS DE RESULTADOS

De las dieciséis obras afectadas por RAS, los tipos de rocas utilizados como árido en la fabricación del hormigón se recogen en la Tabla 2.

Así pues, la tabla de áridos reactivos para el territorio español podría reducirse a los reflejados en la Tabla 3.

Tabla 2. Relación de rocas y componentes reactivos encontrados en los áridos de las obras estudiadas.

Obra	Roca	Componente reactivo
Áridos de reacción lenta		
1	Granodiorita y monzogranito	Cuarzo deformado y fracturado
2	Granodiorita	Cuarzo deformado y fracturado
3	Granitos, gneises, esquistos y diabasas	Cuarzo con extinción ondulante
4	Pizarras	Cuarzo deformado
5	Granito	Cuarzo con extinción ondulante
6	Granodiorita	Cuarzo con extinción ondulante
7*	Granodiorita	Cuarzo con extinción ondulante
8*	Granitos	Cuarzo con extinción ondulante
9*	Cuarzoarenita	Cuarzo con extinción ondulante
10	Granitos y granodioritas	Cuarzo con extinción ondulante
11	Granito	Cuarzo deformado y fracturado
Áridos de reacción rápida		
12	Granodioritas	Cuarzo microcristalino y deformado
13	Cuarcitas	Cuarzo micro y criptocristalino
14	Cuarcitas	Cuarzo micro y criptocristalino
15	Cuarcitas	Cuarzo micro y criptocristalino
16	Cuarcitas (aluvial)	Cuarzo micro y criptocristalino

* Estos áridos no han producido un deterioro visible en la obra afectada aunque sí han sido clasificados como reactivos en el laboratorio.

Tabla 3. Relación de rocas reactivas en España con sus componentes reactivos y el número de casos encontrados.

Rocas		Componente reactivo
Sedimentarias (2 casos)	Cuarzoarenita (1) Aluvial compuesto por cuarcitas (1)	Cuarzo deformado, recristalizado, microcristalino. Cemento silíceo y/o calcedonia
Metamórficas (6 casos)	Cuarcita (3) Esquisto (1) Pizarra (1) Gneis (1)	Cuarzo deformado, recristalizado, micro y criptocristalino
Ígneas (12 casos)	Granito (5) Granodiorita (6) Monzogranito (1)	Cuarzo con extinción ondulante, fracturado, recristalizado con bordes suturados Cuarzo microcristalino

6 CONCLUSIONES

El estudio realizado demuestra la relación directa existente entre geología y patología por reacción álcali-sílice: todas las obras afectadas se encuentran situadas en la España silícea (zona occidental de la Península) y ubicadas en macizos con riesgo potencial de reactividad, de acuerdo con los datos consultados en los correspondientes mapas geológicos.

El estudio petrográfico de los materiales a emplear en la fabricación del hormigón debe empezar con un estudio exhaustivo del mapa geológico de la zona y de su memoria, ya que esta herramienta nos provee de valiosa información de una forma rápida y eficaz.

Desafortunadamente se ha detectado ciertas carencias en algunos de los mapas geológicos consultados, ya sea por ser muy escueta la información señalada (sólo descripción estratigráfica y ausencia de descripción petrográfica) o porque no se encuentran editados.

De acuerdo con el análisis de la información disponible, en España son mayoritarios los casos de RAS lenta (once diagnosticados) en comparación con los ocasionados por la RAS rápida (cinco diagnosticados).

La RAS lenta, más frecuente, tiene especial incidencia en la mitad norte occidental de la geografía española, asociada a los macizos de granitos y granodioritas de estas zonas. El componente reactivo en estas rocas es el cuarzo deformado, fracturado y con extinción ondulante.

La RAS rápida se presenta fundamentalmente en obras situadas en la zona centro-occidental de la Península, relacionada con macizos de cuarcitas metamórficas y cuarzoarenitas. El componente reactivo es el cuarzo micro o criptocristalino, este último en ocasiones en forma de cemento silíceo.

AGRADECIMIENTOS

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Evaluación de la seguridad hidrológica de presas en explotación. Un enfoque alternativo a los planteamientos maximalistas

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ABSTRACT: When re-evaluating design flood considerations in existing dams in order to satisfy the increasingly demanding Safety Regulations, unless an extraordinary episode has taken place in this period, hydrological records corresponding to the lifetime of the structure are usually not taken into account, especially if this information cannot be expressed numerically. In this paper, a case study is presented in which, by means of Bayesian analysis, hydrological information belonging to the operation period is incorporated in the design flood assessment. In this particular case, the evidence added in the evaluation was the known fact that a flood flow assigned to a low return period had not been exceeded in the 30 years of dam operation.

RESUMEN: A la hora de evaluar las necesidades de adecuación de una presa en explotación a las nuevas condiciones de seguridad, en lo que a respuesta ante avenidas se refiere, salvo que se lleguen a producir situaciones excepcionales, suele despreciarse la información hidrológica correspondiente al periodo transcurrido desde la puesta en servicio de la misma, especialmente si ésta es no-numérica. En esta comunicación se presenta un caso real en el que, por aplicación del método bayesiano, se hace un uso intensivo de la información de la historia de la explotación, que, concretamente en el caso presentado, consiste en la constancia de la no superación en 30 años de un caudal asociado a un periodo de retorno reducido.

1 INTRODUCCIÓN

Un concepto que cada vez toma más cuerpo en relación con la seguridad de las grandes infraestructuras como son las presas es el de su consideración como proceso. Las presas no son seguras o no seguras en sí mismas sino que están sometidas a un proceso según el cual cada vez lo son en mayor (mantenimiento) o menor (envejecimiento) medida.

Esto se ve acompañado con la evolución natural de la normativa, que cada vez se va haciendo más conceptual y menos concreta y, simultáneamente, cada vez resulta más exigente.

Este proceso de exigencia creciente de seguridad está teniendo lugar actualmente en España, lo que está poniendo de manifiesto algunas cuestiones que debe considerarse forman parte de lo que podría entenderse como conservación o rehabilitación, entendida esta no tanto como la necesidad de recuperar las características originales de la presa, que pudieran haberse perdido, como en cuanto a la adaptación de la estructura existente a los nuevos requisitos normativos (que traducen o, al menos deben hacerlo, las exigencias sociales).

Por razones obvias, uno de los aspectos más relevantes en relación con la adaptación es el relativo a la seguridad hidrológica, que es el que se aborda en la presente comunicación.

2 PARTICULARIDADES DE LA SEGURIDAD HIDROLÓGICA EN PRESAS EXISTENTES

Quizá el aspecto más relevante y singular de la seguridad hidrológica es que es una cuestión que se mueve en el campo de las acciones más que en el del comportamiento (es semejante probablemente el aspecto relativo a la seguridad sísmica). Mientras que la seguridad estructural o la geotécnica tienen fundamentalmente que ver con la respuesta de la infraestructura a unas situaciones que una presa en explotación ya ha vivido y, por tanto, puede conocerse su respuesta a partir del análisis de su comportamiento, la seguridad hidrológica se apoya en la definición de las avenidas que deben ser consideradas para evaluar el margen de seguridad del que se dispone. En otros términos, la pregunta fundamental es: ¿qué avenidas deben ser consideradas?, y, respecto a esto, el comportamiento

pasado de la presa, al menos aparentemente, no ofrece ningún dato relevante.

Mientras que el paso del tiempo en explotación va reduciendo la incertidumbre en determinadas cuestiones de comportamiento (aunque aparecerán otras incertidumbres, relativas al envejecimiento, por ejemplo) y permite con casi seguridad que se pueda considerar una presa como más segura en el mismo momento que ha superado el primer llenado de manera satisfactoria, en relación con las avenidas este primer llenado no aporta nada. La incertidumbre se mantiene en toda su crudeza.

Mientras que podemos estar más confiados acerca de la seguridad estructural de una presa ya en explotación que ha seguido un buen comportamiento, respecto a la seguridad hidrológica difícilmente puede defenderse algo semejante.

3 LA SEGURIDAD HIDROLÓGICA. PROCESO HISTÓRICO EN ESPAÑA

3.1 *Antecedentes*

Si se acepta que el diseño "científico" de presas comienza en el final del siglo XIX, puede afirmarse que en estos inicios la forma de establecer los requisitos hidrológicos se basaba exclusivamente en la experiencia, haciendo uso bien de fórmulas empíricas o bien aplicando simplemente un factor multiplicador al caudal que se estimase como máximo conocido. Como consecuencia de ello, existía una gran heterogeneidad en los resultados (y en la seguridad).

La rotura (1959) de la presa de Vega de Tera dio lugar a la elaboración de una norma de seguridad ("Instrucción para el Proyecto, Construcción y Explotación de Grandes Presa" de 1962–67) que intentaba poner al día el saber hacer en el campo de las presas y obligar a su aplicación.

Aun cuando la interpretación de las causas de la rotura no implicaba cuestiones hidrológicas, estos aspectos quedaron recogidos de una manera detallada, recurriendo para ello a prescribir el empleo de métodos estadísticos, quizá por influencia de la entonces reciente publicación de la teoría de extremos de Gumbel (1959).

Como se ha dicho, en el citado texto se establece normativamente la utilización de métodos estadísticos, lo que, considerando la escasa densidad de la red foronómica con series de datos de una extensión mínima, realmente conducía a recurrir a aproximaciones hidrometeorológicas a partir de datos pluviometricos sobre series de precipitaciones máximas anuales en 24 horas. Se exigía la consideración de la avenida de 500 años de "periodo de retorno", sobreentendiéndose que el periodo de retorno aplicaba a la precipitación en 24 horas y que, a partir de ella, el proceso seguía criterios conservadores, lo que equivalía, en la práctica, a seleccionar la duración pésima del aguacero de entre todas las que se asociaban a la de 24 horas seleccionada.

No se establecían requisitos mínimos en cuanto a la cantidad ni a la calidad de los datos susceptibles de ser utilizados y no se hacía explicita ninguna necesidad de adoptar un nivel de conservadurismo determinado. De hecho, la práctica habitual ha sido la de utilizar el valor esperado de la estimación, sin aplicar factor de seguridad alguno, siendo el criterio más general el de introducir el conservadurismo en cada una de las tomas de decisión necesarias en el proceso.

Visto desde la perspectiva presente, entonces se disponía de unas herramientas pobres, lo que conducía a que el cálculo necesariamente fuera simple y conservador.

Al ir mejorando el conocimiento y el volumen de información disponible y, como consecuencia, aproximándose la hidrología a lo que pudiera ser una ciencia, los cálculos se van precisando y desaparecen determinados factores implícitos de seguridad (seguridades indirectas), pero, sorprendentemente, se mantiene el requisito de diseñar para hacer frente a avenidas de 500 años de periodo de retorno. Ante esta situación, muchos profesionales consideran que la seguridad hidrológica puede ser escasa y, sobre todo, heterogénea para el conjunto de las presas. Se utilizan distintos subterfugios para aumentar la seguridad sin, aparentemente, contradecir a la normativa.

3.2 *Los tiempos modernos*

En 1982 rompe la presa de Tous y, esta sí, la rotura se asocia a un vertido sobre coronación; se cuestiona ya de una manera explícita el tratamiento que se da a la seguridad hidrológica. Se inicia un proceso (tremendamente largo) para la elaboración de una nueva normativa, que culmina con la publicación (1995) de un nuevo Reglamento Técnico.

Este Reglamento es una norma muy abierta y muy poco concreta, en la línea que entonces era práctica habitual en el mundo. Marca criterios y responsabilidades pero deja indefinidos los niveles de seguridad a alcanzar. Aparecen los criterios de avenidas extrema y de proyecto y se introducen expresamente las revisiones periódicas de seguridad, si bien no se establece el cómo controlar y evaluar esas revisiones desde la Administración.

La desaparición de la referencia explícita a un periodo de retorno determinado (sea el previo de 500 años u otro que le sustituya) produce una añoranza que reclama la existencia de recetas concretas y simples. Ante esta situación aparece una guía técnica que, sin ningún tipo de análisis,

propugna el mantenimiento del empleo del concepto de periodo de retorno y recomienda la utilización, como avenida de proyecto, de la correspondiente a 1.000 años de periodo de retorno (en el caso de presas que tengan un mínimo nivel de riesgo agua abajo).

Pese a tratarse sólo de recomendaciones, los criterios se adoptan y se produce su aplicación automática en las presas en proyecto y, también, en las presas en explotación. Es difícil adoptar criterios distintos a los establecidos en una guía, si se tiene en cuenta que, en caso de accidente, el poder judicial deberá ser el que determine las responsabilidades, incluso penales.

En paralelo a esto, la realización de las revisiones periódicas exigidas por la nueva normativa ha puesto de manifiesto la existencia de un buen número de presas en explotación, construidas al amparo de una legislación anterior y que, cumpliendo esta y no incumpliendo la nueva, se ven cuestionadas por los criterios de una guía no oficial. Ante esta situación de ausencia de criterios consistentes de evaluación surge la cuestión de qué hacer en cada caso concreto.

3.3 *La situación actual*

La situación existente actualmente en España en relación con la seguridad hidrológica de las presas es realmente compleja, por cuanto pudiera llegar a considerarse que existe un buen número de presas que, desde la entrada en vigor del Reglamento Técnico de 1995 (o al menos desde la realización en cada presa de la primera revisión de seguridad exigida), están "fuera de la ley" y debieran ser consideradas inseguras frente a unos requisitos que no tienen carácter legal.

Surge por tanto la necesidad de abordar un proceso de organización de la normativa en el que estamos actualmente inmersos, pareciendo clara la existencia de un sentir entre los técnicos que considera que la aplicación indiscriminada y ciega de los criterios conduce a que determinadas presas razonablemente seguras deban ser remodeladas completamente, esencialmente en lo relativo a la seguridad hidrológica.

4 UN CASO CONCRETO

4.1 *Características principales*

Con el fin de ilustrar alguno de los intentos que se están desarrollando para hacer frente a la problemática planteada se analiza en el presente texto el caso de una presa concreta.

Se trata de una presa (cuyo nombre preferimos no citar) situada en el Pirineo por encima de la cota 1.000, con una cuenca vertiente reducida (200 km^2) y a la que se asocia un tiempo de concentración de entre 1 y 2 horas (cuenca de respuesta rápida). Fue construida antes de la entrada en vigor completa de la Instrucción de 1962–67. El volumen del embalse que cierra es suficientemente pequeño como para poder considerar irrelevante su capacidad de laminación, incluso a embalse vacío.

La capacidad del aliviadero es de unos 600 m^3/s (estimación de la avenida de proyecto en el momento de su redacción).

4.2 *La revisión*

En el curso de la primera revisión general de seguridad a que se ha visto obligada, se ha realizado un estudio hidrológico completo siguiendo el método hidrometeorológico a partir de datos de lluvia en 24 horas extrapolados mediante la ley SQRT-ET max (después de pruebas de otras distribuciones) y convertidos a duraciones distintas mediante formulas de ámbito general. No hay datos de aforos así como tampoco hay datos de entradas al embalse a escala mayor que diaria y aun esta serie es incompleta.

Dado que no existe información alguna en relación con la distribución de los aguaceros (sólo se dispone de datos de pluviómetros, no de pluviógrafos), siguiendo los criterios lógicos de prudencia, se adopta para cada periodo de retorno en precipitaciones en 24 horas la duración pésima.

Como no podía ser de otro modo, se cumple inexorablemente la ley de caudales crecientes en estudios sucesivos y el estudio arroja los resultados que se presentan en la Figura 1.

La aplicación automática de los resultados anteriores conduce a la necesidad de abordar un cambio sustantivo en la presa y sus órganos de desagüe, lo que contradice un sentimiento (irracional) asentado en los evaluadores en el sentido

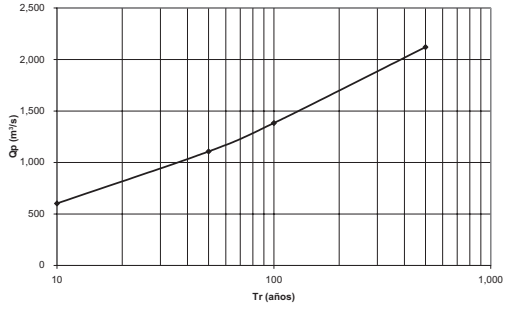

Figura 1. Caudales punta de avenida de entrada a la presa objeto de estudio para distintos periodos de retorno según los resultados del modelo hidrometeorológico elaborado.

de que la situación no puede ser tan grave, ya que lleva la presa más de 30 años en explotación y no ha superado en ningún momento la capacidad del aliviadero. Si se dan por buenos los resultados del estudio, el caudal de diseño se correspondería con el de una avenida asociada a un periodo de retorno de 11 años aproximadamente.

4.3 *Primera aproximación*

Si en lugar de referirnos al resultado final asociado a la duración pésima se da un paso atrás y se observan los caudales punta que corresponden a los mismos periodos de retorno pero expresados para una gama más amplia de duraciones de aguacero se obtienen lo resultados que se reflejan en la Figura 2.

En ella se aprecia la gran diferencia que existe entre considerar duraciones reducidas o mayores. Mientras que para duraciones de aguacero comprendidas entre 1 y 4 horas, la punta de caudal para un periodo de retorno de 500 años está en el entrono de los 2.000 m^3/s, en aguaceros de duración superior a las 12 horas apenas llegan a superarse los 1.000 m^3/s.

Este comportamiento es el esperable en una cuenca de respuesta hidrológica rápida. Precipitaciones de reducida duración son suficientes para garantizar que la circulación superficial de todos los puntos de la cuenca llegará de manera simultánea a la sección de cauce donde se localiza el embalse. Al ir estas duraciones cortas de aguacero asociadas a elevadas intensidades, se obtienen como resultado avenidas de escaso volumen de hidrograma pero caudal punta muy elevado. A medida que se consideran mayores duraciones de aguacero la intensidad de precipitación disminuye. Como el requisito de la contribución de la totalidad de la cuenca vertiente a la generación simultánea de escorrentía ya quedaba garantizado para la hipótesis anterior, esta nueva situación no comporta ventaja respecto a aquella, por lo que la punta de avenida se reduce de manera muy notable.

Como consecuencia de lo anterior puede pensarse que la selección de la condición pésima es, al menos, una de las causas fundamentales de lo elevado del caudal. Si se pudiese establecer que la duración del aguacero más frecuente en fenómenos extraordinarios, por ejemplo, fuese superior a 12 horas, se podría defender la validez de un aliviadero unos 1.000 m^3/s para el criterio de los 500 años.

4.4 *Posibilidades de actuación*

A la vista de la muy notable diferencia existente entre el caudal de diseño y el estimado a partir de la modelización desarrollada con posterioridad, surge la necesidad de estudiar acciones correctoras que permitan mejorar las condiciones de seguridad de la presa. En ese sentido, siempre que se plantean este tipo de cuestiones es recomendable (incluso inevitable) tener presente el otro punto de vista, esto es, además del ¿qué hay que hacer? el ¿qué se puede razonablemente hacer?

Evidentemente, se podría abordar una remodelación total del aliviadero, modificándolo hasta alcanzar las nuevas necesidades planteadas, pero antes que llegar a ese extremo es conveniente analizar soluciones menos drásticas.

Por una parte, incorporando a la coronación un pretil impermeable, al modo de un botaolas, podría apurarse la capacidad de desagüe del aliviadero, permitiendo mayores alturas de lámina sin riesgo a producir vertidos por coronación.

En el caso de que esta alternativa no fuese suficiente, al tratarse de una presa de gravedad, podría, asimismo, permitirse un cierto vertido sobre coronación, siempre y cuando tanto la altura de lámina como la duración de este sobrevertido se encontrasen dentro de unos límites razonables.

El significado de estas posibilidades se pone de manifiesto en la Figura 3, en la que se refleja,

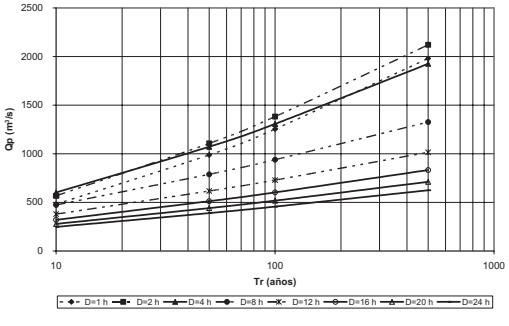

Figura 2. Relación caudal punta de avenida—periodo de retorno para distintas duraciones de aguacero.

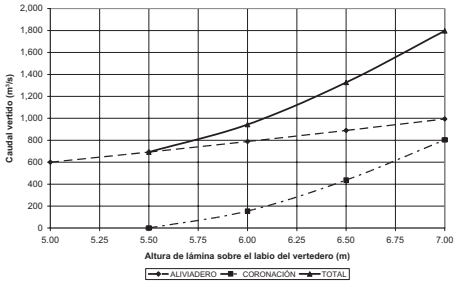

Figura 3. Caudales de suelta por aliviadero y por sobrevertido por coronación en función de la altura de lámina.

para la presa objeto de estudio, una aproximación obtenida mediante ecuaciones simplificadas del caudal vertido para distintos niveles de embalse, bien por el aliviadero, por coronación o por ambos simultáneamente.

Se aprecia que un caudal de unos 700 m^3/s puede conseguirse prácticamente sin actuación, mientras que vertidos de 900–1.000 m^3/s serían alcanzables sin grandes actuaciones (pequeña adecuación del vertedero y sobreelevación de coronación de 0,5–1 m). Finalmente, podría llegar a alcanzarse un caudal de 1.300–1.500 m^3/s si se admite, además de la adecuación del vertedero, un vertido sobre coronación de, como máximo, un caudal específico de alrededor de 2 $m^3/s/m$.

Téngase en cuenta que los casos de grandes caudales corresponden a duraciones de aguacero reducidas, lo que permite establecer que el vertido por coronación estaría también acotado en el tiempo.

5 ANÁLISIS

5.1 *Planteamiento inicial*

Más allá del estudio de la capacidad hidráulica del aliviadero, se apuesta por realizar un análisis que permita establecer un diagnóstico sobre cuales son los escenarios hidrológicos para los que habría que garantizar la respuesta de la presa en términos de su seguridad hidrológica.

Como ya se ha indicado, la presa objeto de estudio lleva más de 30 años en explotación y, hasta la fecha, se mantiene en servicio sin incidencias hidrológicas serias conocidas, lo que podría interpretarse como que su aliviadero tiene capacidad suficiente como para laminar, al menos, las avenidas que hasta el día de hoy hayan podido producirse.

Sin embargo, esta afirmación va en contra de los resultados proporcionados por la modelización realizada, de acuerdo con la cual la probabilidad de haber superado el caudal de diseño del aliviadero, al menos una vez en los últimos 30 años, sería superior a 0,94.

A la vista de esta aparente contradicción, surge el planteamiento de que la información que proporciona el conocimiento hidrológico adquirido en los años de explotación de la presa debería de alguna manera integrarse a la modelización desarrollada. La posibilidad apuntada, conceptualmente podría asimilarse al habitual proceso de ajuste de los parámetros de un modelo hidrometeorológico para hacerlo coherente con las referencias de caudal de puntos de aforo de las que se pudiera disponer.

Para la realización de esta tarea, el análisis se plantea por aplicación del método bayesiano bajo los siguientes supuestos:

- Como parámetro de ajuste a estimar se considera la duración del aguacero y la correspondiente asignación de probabilidades a cada una de estas duraciones. En este parámetro (como se ha visto, fundamental) se consideran incluidas también las restantes variables de un orden inferior.
- Se considera que la duración puede adquirir valores de hasta 24 horas. Esto tiene un algo de arbitrario pero, dadas las características de la cuenca no sería especialmente significativo el hecho de ampliar el intervalo.
- Se acepta la validez de los modelos estadísticos que, para una duración de aguacero dada, permiten asociar periodo de retorno con caudal punta.
- Como información de trabajo se establece que no se ha superado el caudal de diseño del aliviadero (600 m^3/s) en los 30 años de vida de la presa.

(Se señala, por su trascendencia conceptual, que en toda la comunicación, cuando se habla de periodo de retorno de la avenida se refiere a la variable caudal punta de la avenida).

5.2 *Planteamiento metodológico*

La aplicación del modelo bayesiano se basa en establecer una distribución de partida del parámetro poblacional (la duración de aguacero en nuestro caso). Esta información, incorporada a priori, puede provenir bien de un conocimiento objetivo de la distribución de frecuencias de la variable en cuestión o bien puede quedar establecida por aplicación de un determinado criterio subjetivo. En el caso analizado resulta razonable, en ausencia de datos que permitan caracterizar la distribución temporal de la precipitación, asignar de manera conservadora un mayor nivel de probabilidad a las duraciones en el entorno de la pésima.

Una vez establecida la distribución de probabilidad *a priori* se utiliza la información conocida para dar o restar validez a este supuesto obteniéndose de esta manera una probabilidad condicionada o *a posteriori*. En este caso, el conocimiento de que la presa no ha superado el caudal de diseño del aliviadero desde que entró en explotación es la información adicional disponible que se utilizará para "corregir" la formulación de probabilidad asignada de inicio a las distintas duraciones.

Explicado lo anterior en términos un tanto más pragmáticos, lo que se pretende es, frente a un planteamiento que pudiera invitar a pensar que el hecho de que un suceso considerado como esperable a corto plazo no haya llegado a producirse hasta la fecha implica que la probabilidad de que este vaya a ocurrir de manera inminente aumenta de manera inexorable con el paso del tiempo, una

negación de la hipótesis de partida: si transcurrido un tiempo suficientemente largo no se ha producido una avenida a la que la modelización desarrollada asigna una probabilidad alta en términos relativos, hay argumentos estadísticos suficientemente consistentes como para afirmar que los supuestos de partida (la distribución de probabilidades establecida de antemano) debe ser corregida.

A continuación se recoge la expresión general del teorema de Bayes:

$$P\left(\frac{A_i}{B}\right) = \frac{P\left(\frac{B}{A_i}\right) \cdot P(A_i)}{\sum_{j=1}^{n} P\left(\frac{B}{A_j}\right) \cdot P(A_j)} \qquad (1)$$

En nuestro caso, el suceso A se corresponde con la variable de duración de aguacero. La escala de duraciones se ha discretizado, considerando los intervalos 0–1; 1–2; 2–4; 4–8; 8–12; 12–16; 16–20 y 20–24. P(A) responde por tanto a la distribución de probabilidades de duración de aguacero establecida *a priori*.

Por otra parte P(B) se identifica con la probabilidad de no superación de la capacidad de diseño del aliviadero en los 30 años de historia de la presa.

Por extensión, $P(A_i/B)$ es la probabilidad de que se produzca una duración de aguacero determinada, condicionada al hecho conocido de que no se ha superado el caudal de diseño del aliviadero (probabilidad *a posteriori*, en definitiva es el valor a obtener).

Por último, $P(B/A_i)$ debe entenderse como la probabilidad asignada al hecho de que no se haya superado el antes referido caudal en el tiempo de vida de la presa, de acuerdo con la ley caudal—periodo de retorno correspondiente a una duración determinada A_i, obtenida de los resultados proporcionados por el modelo hidrológico.

5.3 *Cálculo desarrollado*

El cálculo se ha realizado según el siguiente esquema:

A cada uno de los intervalos de duración considerados se les ha asignado, para cada periodo de retorno, el caudal punta medio entre los correspondientes a las dos duraciones extremas de la división.

Se ha establecido, por ajuste estadístico y para cada duración, la relación entre el periodo de retorno (expresado en forma de doble logaritmo) y el caudal punta (la regresión en todos los casos tiene un coeficiente superior a 0,99). La forma de ajuste es una expresión exponencial. Los resultados se reflejan en la figura siguiente.

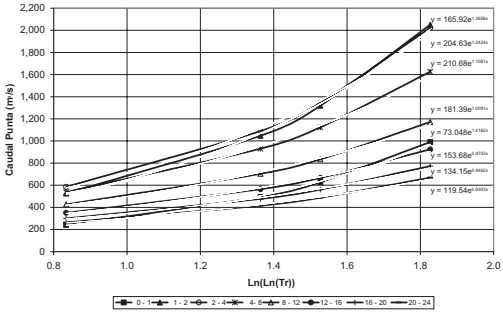

Figura 4. Fórmulas de ajuste entre caudal y periodo de retorno para cada uno de los intervalos de duración establecidos.

De esta manera, queda establecida una relación numérica entre caudal y periodo de retorno asociado para cada uno de los supuestos de duración considerados.

Una vez definidas estas relaciones ya es posible determinar, para cada rango de duraciones definido, la probabilidad de no haber alcanzado un caudal límite en un número determinado de años.

Con estos datos, a partir de una distribución estadística de duraciones considerada de inicio, por aplicación de la relación de Bayes formulada, podrá determinarse la distribución estadística corregida que incorpore el conocimiento adquirido en los años de explotación.

5.4 *Primeros resultados*

Como primera aproximación al problema, se ha aplicado el esquema de cálculo anterior suponiendo como distribución a priori la uniforme.

Esta opción es la recomendada cuando no existe un conocimiento previo del comportamiento de la muestra. Responde al criterio de entregar al modelo la distribución que contiene la menor información posible. Dada nuestro desconocimiento no atribuimos inicialmente mayor probabilidad a ninguna duración del aguacero.

Como resultado de la aplicación del teorema de Bayes, la distribución de probabilidades corregida es la que se incluye en la Figura 5. En ella se aprecia cómo la consideración de la no superación del caudal de diseño del aliviadero en una serie larga de años conduce a incrementar las probabilidades de las duraciones más largas y reducir las asociadas a las duraciones pésimas.

Es relevante conocer la influencia que, en términos de caudal, se produce al variar la distribución de probabilidad asociada a la duración. Para ilustrar esta relación, se ha obtenido el valor esperado (o esperanza matemática) del caudal punta de avenida asociado a un periodo de retorno de 500 años

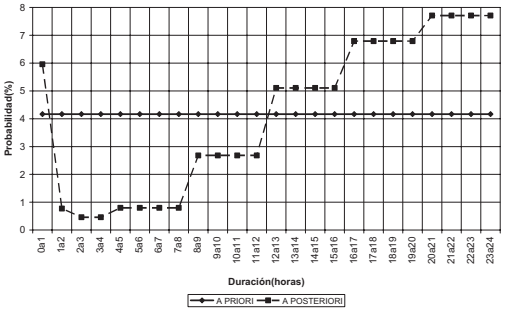

Figura 5. Probabilidad asignada a cada duración, según la distribución a priori y la corregida por aplicación del teorema de Bayes.

Tabla 1. Valor esperado del caudal punta de avenida según la hipótesis de distribución de duraciones considerada.

	Qpunta
	m^3/s
Duración pésima	2.100
Distribución a priori	1.150
Distribución a posteriori	875

para cada una de las distribuciones de probabilidad presentadas. Para completar la evolución, se presenta también el caudal punta correspondiente a la duración pésima (Tabla 1).

A primera vista, el resultado anterior, aplicado de un modo mecánico, resolvería el problema. Con diseñar una solución que permitiera el incremento de la capacidad de vertido por el aliviadero hasta 875 m^3/s las condiciones de seguridad serían razonables. Este caudal de suelta podría alcanzarse sin mayor dificultad. Por ejemplo, bastaría tan solo con admitir una lámina vertiente de 6,5 m en el aliviadero en lugar de los 5 actuales y establecer un pretil de 1 m de altura sobre la cota de coronación.

5.5 *Primera discusión*

Ahora bien, si se profundiza en la evolución del caudal punta esperado según la hipótesis de partida considerada, se aprecia que realmente la incorporación de la información adicional proporcionada (la no superación del caudal de diseño del aliviadero) no se traduce en una reducción drástica del caudal punta esperable, ya que sólo implica una variación desde 1.150 a 875 m^3/s. Es muy superior el salto que se produce entre considerar únicamente la duración pésima o plantear una ley de distribución equiprobable (la reducción es de casi el 100%, pasándose de 2.100 a 1.150 m^3/s).

Y, lógicamente, esta aproximación no parece razonable ya que representa un cambio de metodología que no tiene sustento: en ausencia de datos se opta por adoptar el valor medio en una cuestión relativa a la seguridad de la población. Entendida la seguridad de las presas como un servicio público, ninguna Administración responsable admitiría como expresivo de las condiciones de seguridad el valor medio sin ningún tipo de corrección. Ciertamente puede justificarse el no adoptar el valor pésimo, pero desde luego se hace necesario el introducir un cierto nivel de seguridad adicional respecto a la media de las condiciones.

Aunque sea marginalmente, es importante señalar que adoptar la distribución uniforme es correcto desde el punto de vista del conocimiento del técnico, pero no representa nada desde el punto de vista real. Es perfectamente compatible que la duración más probable de los aguaceros extremos sea de 2 horas por ejemplo con el hecho de que, al no ser esto conocido y en ausencia de otra información, se opte por establecer una distribución uniforme. De hecho, la distribución real de las duraciones puede distar mucho de la uniforme, pero sí puede seguir una distribución de este tipo nuestro conocimiento del sistema real.

Cierto es que el resultado anterior podría ser útil en determinados casos en el sentido de comparar la situación de distintas presas entre sí en términos de seguridad hidrológica (se podría otorgar una prioridad a la actuación sobre la presa a la que se asociase un valor esperado más alto) en relación a la capacidad de su aliviadero, pero en absoluto permite conclusiones en términos absolutos.

5.6 *Un paso más*

Ante la situación expuesta, en España y en este momento, ante la pregunta de cómo se puede seguir se respondería, con bastante generalidad y de una manera casi automática (probablemente sólo por razones de moda) con "la solución es … el análisis de riesgos". Ahora bien, un análisis de riesgos es lo que se está haciendo, teniendo presente que el problema es precisamente establecer la probabilidad que se asigna a una variable fundamental, y esto es lo que utiliza como base el análisis de riesgos tradicional y clásico.

Retomando el hilo del análisis, los resultados anteriores han puesto de manifiesto que considerar a priori una distribución uniforme para la probabilidad de la duración del aguacero deriva en que los resultados que se obtienen resultan poco conservadores. Sin embargo, lo anterior no conduce a cuestionar el análisis bayesiano en sí mismo. Por este motivo se opta por continuar con este análisis, si bien modificando la distribución a priori.

Para ello, y como es habitual, se parte de suponer que existe efectivamente una distribución a priori y que esta puede aproximarse suficientemente por aplicación de una de la familia Beta. Esta familia, de dos parámetros más otro posible de escala, permite representar muchas formas continuas que van desde la uniforme (ambos parámetros iguales a la unidad) hasta distribuciones muy apuntadas.

Los estadísticos de la distribución β(a,b) son (media: μ; moda: m; desviación típica: σ)

$$\mu = a/(a+b) \tag{2}$$

$$\mu = (a-1)/(a+b-2) \tag{3}$$

$$\sigma = (a \times b)\Big/\left[(a+b)^2 \times \sqrt{(a+b+1)}\right] \tag{4}$$

Que existe una distribución estadística para la duración del aguacero, en contraposición fundamentalmente a la posibilidad de que se trate de un valor determinista, es consecuencia de los años de explotación. Aunque se carezca de información numérica sí es conocido que existen grandes aguaceros de distintas duraciones, desde cortas a largas (si todos se moviesen en el entorno de las 2 horas, por ejemplo, el hecho sería conocido).

Un paso más, que permite eliminar un parámetro, es el de suponer, de manera conservadora, que la moda de la distribución se asocia a duraciones cortas (se acepta el valor de 2 horas). Se habla de la moda por cuanto es este estadístico más fácil de caracterizar que la clásica media: en general es más sencillo identificar lo más frecuente que lo medio. Por otra parte, al ser los caudales mayores para las duraciones cortas, el criterio introduce el factor de seguridad adicional que parece necesario. Se trata, por tanto, de un criterio conservador.

A partir de aquí, el proceso consiste en analizar la resultados que se producen partiendo de una distribución a priori de las duraciones del aguacero que, desde la uniforme y teniendo siempre la moda en las 2 horas, van haciéndose más apuntadas, aproximando la media a la moda y reduciendo la desviación típica.

Las analizadas son las que se recogen en la Tabla 2 adjunta.

Algunas de estas distribuciones se reflejan en la Figura 6, en la que se aprecia cómo realmente los valores altos de "b" (distribuciones muy apuntadas) realmente no responden a las hipótesis establecidas, Así, por ejemplo, la distribución β(1,64;8) descarta (probabilidad nula) las duraciones por encima de 20 horas. Incluso la distribución β(1,27;4) puede considerarse como dudosa (asigna a las duraciones por encima de las 16 horas algo menos del 2% de probabilidad).

Tabla 2. Estadísticos de las distribuciones β(a;b) de moda 2 consideradas en el estudio.

a	1,00	1,09	1,18	1,27	1,36	1,45	1,55	1,64
b	1,00	2,00	3,00	4,00	5,00	6,00	7,00	8,00
μ	12,0	8,47	6,78	5,79	5,14	4,68	4,34	4,08
σ	6,93	5,67	4,75	4,10	3,63	3,27	2,99	2,76
m		2,00	2,00	2,00	2,00	2,00	2,00	2,00

El resultado de los cálculos se refleja en la Figura 7, en la se representan las distribuciones a posteriori (líneas continuas) de las duraciones del aguacero que se obtienen a partir de distintas distribuciones a priori (líneas de trazos).

Se aprecia cómo la consideración de la información que se desprende de la explotación hace que sistemáticamente se vean incrementadas las probabilidades de las duraciones a las que corresponden caudales menores a costa de las que corresponde a caudales más altos, en la misma línea que ya se ha expuesto para el caso de distribución a priori uniforme (punto 5.4). Obviamente, la probabilidad a posteriori depende de la a priori considerada; así, la probabilidad que se obtiene para una duración comprendida entre 2 y 3 horas es del 0,46; 1,10 o 2,53% dependiendo que se haya partido de una probabilidad a priori del 4,17; 6,87 o 10,61% respectivamente.

Ahora bien, lo anterior no contesta, al menos directamente, a la pregunta de qué hacer, aunque sí indica el camino.

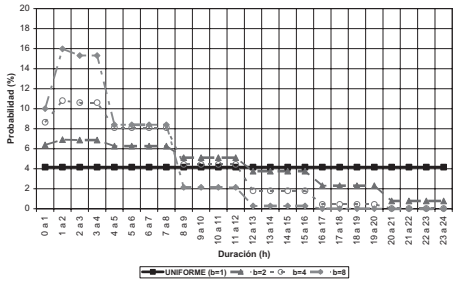

Figura 6. Distintas funciones β de moda 2 horas.

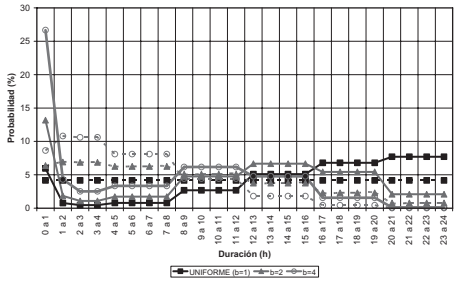

Figura 7. Distribuciones a posteriori obtenidas partiendo de distintas distribuciones a priori (β de moda 2 horas).

Este parte de otra manera de ver los resultados, probablemente más intuitiva y expresiva y que ya conduce a la respuesta buscada, como es la que resume el periodo de retorno (la probabilidad de superación) que corresponde a determinados caudales significativos.

Los resultados se presentan en las Tablas 3 y 4, en las que se reflejan tanto los valores de los periodos de retorno que se asocian a distintos caudales, aceptando distintas distribuciones a priori (tabla 3) y los que les corresponden una vez corregidas estas distribuciones (tabla 4).

Estos mismos resultados se presentan de forma gráfica en la Figura 8, en la que se expresa el caudal punta que se asocia a distintos periodos de retorno para distintas hipótesis de distribuciones de duración del aguacero presupuestas a priori (líneas de trazos) y las que se obtienen a posteriori (líneas continuas). Como referencia también se presenta la misma relación caudal punta-periodo de retorno que corresponde a la duración pésima.

A la vista de estos resultados ya es posible un análisis satisfactorio, que se expone en el punto que sigue.

Tabla 3. Periodos de retorno que corresponden a distintos caudales punta para varias distribuciones β(a;b) de moda 2, distribuciones a priori.

	Parámetro b de la distribución β				
Caudal (m^3/s)	1	2	4	8	Pésimo
600	28	20	16	14	11
800	66	45	34	28	20
1000	140	92	67	54	36
1500	726	459	317	240	144
2000	3047	1891	1272	930	488

Tabla 4. Periodos de retorno que corresponden a distintos caudales punta para varias distribuciones β(a;b) de moda 2, distribuciones a posteriori.

	Parámetro b de la distribución β				
Caudal (m^3/s)	1	2	4	8	Pésimo
600	65	42	29	24	11
800	203	113	70	51	20
1000	544	277	155	105	36
1500	3934	1790	876	520	144
2000	18943	8268	3814	2113	488

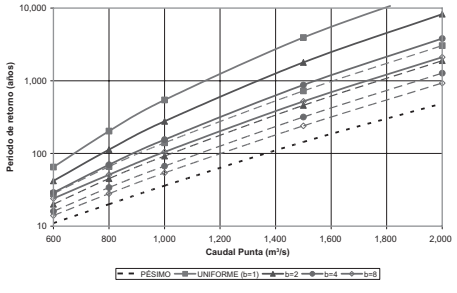

Figura 8. Relación caudal punta-periodo de retorno para distintas distribuciones de duración del aguacero. Comparación de resultados a priori y a posteriori.

5.7 *Análisis final*

Si se clasifican las posibilidades de actuación en función de su importancia, pueden plantearse tres niveles: no actuación, remodelación completa y actuación limitada.

La no actuación no es posible, ya que la capacidad del aliviadero sería de 600 m^3/s (como máximo de 700), lo que corresponde a periodos de retorno mayores de los 11 años que se asocian a la consideración de la duración pésima pero excesivamente bajos tanto en la a priori como a posteriori (en la situación más optimista no se llega a los 100 años de periodo de retorno).

La remodelación completa no parece necesaria. Querría decir ir a buscar un caudal de del orden de 2.000 m^3/s, lo que se asocia a una probabilidad baja de superación en todas las condiciones (en todas las distribuciones se superan los 2.000 años de periodo de retorno, llegando a superar los 10.000).

Adecuar el aliviadero en el entorno de los 1.000–1.500 m^3/s parece razonable (preferiblemente en el entorno de los 1.500). Los periodos de retorno que corresponden se mueven en el entorno debido. Por otra parte, dado que previamente (apartado 4.4) se ha establecido que el tope para poder abordar sólo una remodelación limitada corresponde a un caudal de 1.300 m^3/s, parece razonable adoptar este valor como objetivo.

Es interesante notar que la conclusión anterior no hubiese sido posible en ausencia de los datos que ha proporcionado la explotación, que en este caso son muy extremos.

6 CONCLUSIONES

La propia explotación, aun en ausencia de valores puramente numéricos, puede aportar información útil para el análisis de la seguridad hidrológica de presas con una larga historia, si bien la información aportada no tiene la calidad suficiente como para ser adoptada de una manera mecánica, por lo que es preciso un análisis detallado y cuidadoso, adecuado al caso concreto de que se trate y de muy difícil normalización.

En el caso concreto de una presa con una larga historia de explotación y que muestra una holgura importante en su comportamiento hidrológico en relación con los criterios que se utilizaron en proyecto, la comunicación ha puesto de relieve cómo la aplicación del análisis bayesiano ha permitido acotar el nivel de la actuación que es preciso acometer para conseguir su adecuación a las exigencias actuales de seguridad, evitando planteamientos maximalistas.

En cualquier caso, también resulta evidente que la decisión relativa al qué hacer es delicada y su responsabilidad (al afectar las consecuencias a la población) no puede descansar sobre los hombros de un solo técnico, por muy capacitado que este se encuentre. Es precisa la existencia de un organismo supervisor con independencia, capacidad y competencia suficientes específicamente dedicado a las cuestiones de seguridad de presas.

Interesa poner de manifiesto que si bien en el ejemplo presentado se ha utilizado sistemáticamente la duración del aguacero como parámetro "incógnita", este pudiera haber sido cualquier otro respecto al cual el resultado determinista sea de una sensibilidad elevada (otra posibilidad sería en este caso el estado de humedad previo de la cuenca). El resultado sería semejante al aquí expuesto (la conclusión sería que la probabilidad de un estado de humedad alto en la cuenca sería baja). Incluso puede plantearse la consideración simultánea de más de una variable.

Desde otro punto de vista, es claro que el análisis realizado puede extenderse a aspectos clásicos del diseño, como pueden ser los relativos a hacer converger los métodos foronómicos y los hidrometeorológicos, cuando ambos sean posibles (apoyo en datos foronómicos para tomar decisiones relativas a parámetros utilizados en los métodos hidrometeorológicos, por ejemplo).

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Forecasting of dam flow-meter measurements using Artificial Neural Networks

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ABSTRACT: Dam monitoring requires first data collection and second data analyzing. This second part of monitoring needs to use models which reproduce dam behavior. Nowadays there are two methods widely used: the statistical method and the finite elements method. Both methods estimate dam behavior using environmental variables, such as water level in the reservoir and ambient temperature. This paper presents a method that uses Artificial Neural Networks (ANNs) for determining dam seepage. The method is applied to La Baells dam, an arch dam situated in Barcelona (Spain).

1 INTRODUCTION

The main goal of dam monitoring is to ensure the safety of dams. For it, data are collected and then analyzed. The recorded data are usually seepage flow, ambient temperature, water level in the reservoir, dam displacements, foundation uplift pressure, relative movement across joints, between others. Data are analyzed and compared to models. Currently, these models are developed using the Finite Element Method (FEM) or statistical models.

FEM models try to reproduce the interaction between dam and its foundation under actions. But the interaction is very complex and nowadays its behavior is not properly known. As a result, simplifications and hypotheses have to be made which lead to less accurate models.

Statistical models try to predict dam behavior using a linear combination of complex functions. So this model consists on a sum of functions multiplied by coefficients which are usually determined by the least squared method. Hypotheses have to be made about the most appropriate functions.

Opposite to this, models developed using ANNs do not involve simplifications or hypotheses. ANNs have been applied in almost all branches of science and engineering (Nedushan and Chouinard 2003). Flood et al (Flood et al. 1994), Waszczyszyn (Waszczyszyn 2006), Meireles et al (Meireles et al. 2003) or Adeli (Adeli 2001) have written some overviews about ANNs applications to engineering. ANNs have been applied to seismic hazard prediction, settlements of shallow foundations estimation, river flow prediction, finite elements mesh generation. ANNs have been used in dam engineering for: predicting water absorption in the sealing process in a dam ground curtain, inverse analysis for health assessment of concrete dams, predicting relative crest settlement of concrete-faced rockfill dams (Kim and Kim 2008), real time analysis of dam monitoring data (Nedushan and Chouinard 2003) and for predicting water level in piezometers of an earthfill dam (Tayfur et al. 2005).

Several ANNs features have let this purpose. Some of them are:

- ANNs have the skill to learn and generalize from examples. Moreover, ANNs are able to recognize complex relationships between input and output patterns when a sufficient number of data are given.
- They can cope with problems where the input data is fuzzy, discontinuous or incomplete.
- ANNs are very useful in problems where the relationships between input and output patterns are unknown. As a result, they can model unknown phenomenon. They have been used successfully to perform complex function approximations.
- ANNs can calculate solutions rapidly.

But ANNs have drawbacks. Some of them are that ANNs are a black box and its architectural design is based on trial and error method.

1.1 *Aims of the research*

The main aim of this research is to develop a simpler and more accurate dam seepage model than the developed by statistical procedures, currently used for dam monitoring analysis.

The global propose is to obtain a model for predicting the seepage flow forecast at every flow meter. This model is based on an ANNs and is applied to

a test case, La Baells dam. Currently in this dam, a statistical model is used to analyzed monitoring data. The ANN seepage model is implemented by a program ming language, so it can be used under a user-friendly computer program.

2 INTRODUCTION TO ANNs

The ANN used in this research was a Multilayer Perceptron (MLP) (Figure 2). It is a parallel computing structure. It is composed by simple computational units, called neurons. Neurons are organized in layers. Layers are connected only with the previous layer, where they receive information. There are no connections between neurons in the same layer. This structure tries to imitate biological neurons (Haykin 1999).

2.1 *Neuron operations*

For a given j artificial neuron (Figure 1) $\vec{X} = [1, x_1, x_2, \ldots, x_i, \ldots, x_N]$ is the input data, where x_i is the i^{th} component. The x_0 input is assigned the value 1, which makes it a bias input.

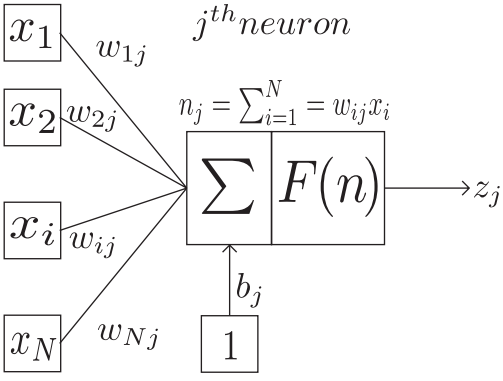

Figure 1. An artificial neuron.

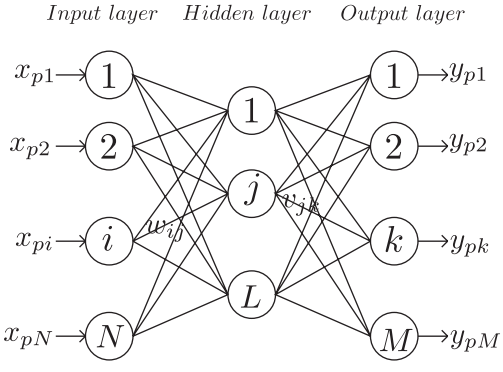

Figure 2. An MLP structure.

$W_j = [w_{0j}, w_{1j}, w_{2j}, \ldots, w_{ij}, \ldots, w_{Nj}]$ is the j^{th} row of the weight matrix, where w_{ij} is the j^{th} neuron weight for the i^{th} component of input data. w_{0j} is a particular weight, $w_{0j} = b_j$ is the bias for the j^{th} neuron. The output of the j^{th} neuron is:

$$z_j = F\left(\sum_{i=0}^{N} w_{ij}x_i\right) \tag{1}$$

where F is the transfer function.

2.2 *ANN operations*

For a given ANN (Figure 1) $\vec{X}_p = [1, x_{p1}, \ldots, x_{pi}, \ldots, x_{pN}]$ is the p^{th} input data, where x_{pi} is the i^{th} component or the input to i^{th} input neuron. The x_0 input is assigned the value 1, which makes it a bias input. w_{ij} is the weight between i^{th} input neuron and the j^{th} hidden neuron. The weights simulate the synapses between neurons, as in biological neurons. v_{jk} is the weight between j^{th} hidden neuron and k^{th} output neuron. n_{pj} is the input to j^{th} hidden neuron, and n_{pk} is the input to k^{th} output neuron, for a p pattern. z_{pj} is the output from j^{th} hidden neuron and y_{pk} is the output from k^{th} output neuron, for a p pattern.

For a given ANN composed of three layers, N neurons in the input layer, L neurons in the hidden layer and M neurons in the output layer, the weight matrix **W** is:

$$\mathbf{W} = \begin{pmatrix} b_1 & b_2 & \cdots & b_j & \cdots & b_L \\ w_{11} & w_{12} & \cdots & w_{1j} & \cdots & w_{1L} \\ \vdots & \vdots & \ddots & \vdots & \ddots & \vdots \\ w_{i1} & w_{i2} & \cdots & w_{ij} & \cdots & w_{iL} \\ \vdots & \vdots & \ddots & \vdots & \ddots & \vdots \\ w_{N1} & w_{N2} & \cdots & w_{Nj} & \cdots & w_{NL} \end{pmatrix} \tag{2}$$

where b_j is the j^{th} hidden neuron bias. And the weight matrix **V** is:

$$\mathbf{V} = \begin{pmatrix} b_1 & b_2 & \cdots & b_k & \cdots & b_M \\ v_{11} & v_{12} & \cdots & v_{1k} & \cdots & v_{1M} \\ \vdots & \vdots & \ddots & \vdots & \ddots & \vdots \\ v_{j1} & v_{j2} & \cdots & v_{jk} & \cdots & v_{jM} \\ \vdots & \vdots & \ddots & \vdots & \ddots & \vdots \\ v_{L1} & v_{L2} & \cdots & v_{Lk} & \cdots & v_{LM} \end{pmatrix} \tag{3}$$

where b_k is the k^{th} output neuron bias.

For a given $\vec{X}_p$ input, n_{pj} is calculated by:

$$n_{pj} = \sum_{i=1}^{N} w_{ij}x_{pi} = \mathbf{X} \cdot \mathbf{W} \tag{4}$$

And z_{pj} is given by:

$$z_{pj} = F(n_{pj});\ \vec{Z}_p = (1, z_{p1}, z_{p2}, \ldots, z_{pj}, \ldots, z_{pL}) \quad (5)$$

where $F(n)$ is the transfer function. The most common transfer function in hidden layers is the tan-sigmoid function, which is shown in equation 6.

$$F(n) = 2/(1 + exp(-2n)) - 1 \quad (6)$$

Neurons situated in the output layer usually have linear transfer functions. For the proposed application, tan-sigmod transfer function may be used, due to the best experiment results were reached by this.

n_{pk} is determined by:

$$n_{pk} = \sum_{j=1}^{L} v_{jk} z_{pj} = \mathbf{Z} \cdot \mathbf{V} \quad (7)$$

And the ANN output y_p is calculated by:

$$y_{pk} = F(n_{pk})$$
$$Y_p = (y_{p1}, y_{p2}, \ldots, y_{pk}, \ldots, b_{pM}) \quad (8)$$

2.3 *Learning process*

The learning process or training consists on determining the weight values which minimize the difference between the ANN output and the desired output, called the goal, for a given input-output data. The trained is fulfilled by an algorithm. Usually MLP are training by backpropagation algorithm. This algorithm needs an input-output data set, from where, in an informal way, "the algorithm obtains the information to teach the ANN".

3 SEEPAGE PHENOMENON

Seepage through a fractured rock mass can be approached with different models. Current approaches to modeling fractured rock are the equivalent continuum model, the fracture network model and the double media model. The last two models are more reliable, but in both cases it is difficult to acquire enough parameters to accurately represent the geometric characteristics and physical properties of the fissures (Li et al. 2008). The equivalent continuum model does not need the specific position and hydraulic property of every fissure. For this reason, it will be use to explain the seepage phenomenon and to determine the variables it depends on. This model was used for determining seepage through a fracture rock mass in the upper reservoir of Pushihe pumped-storage power station in China (Li et al. 2008).

3.1 *Seepage mathematical model*

For a given groundwater system, being heterogeneous, anisotropic and having steady flow, it can be modeled using the following partial differential equation:

$$\frac{\partial}{\partial x}\left(K_{xx}\frac{\partial H}{\partial x}\right) + \frac{\partial}{\partial y}\left(K_{yy}\frac{\partial H}{\partial y}\right) + \frac{\partial}{\partial z}\left(K_{zz}\frac{\partial H}{\partial z}\right) = 0;\ (x,y,z) \in \Omega \quad (9)$$
$$H(x,y,z)\,|\,s_1 = H_1(x,y,z);\quad (x,y,z) \in S_1$$
$$K_n \frac{\partial H}{\partial \vec{n}}\bigg|_{S_2} = q(x,y,z);\qquad (x,y,z) \in S_2$$

where Ω is the simulation volume; (x, y, z) are the location coordinates; K_{xx}, K_{yy} and K_{zz} are the hydraulic conductivity components along x, y and z directions (providing that the principal hydraulic conductivity axes are consistent with the coordinate axes); S_1 is the constant head boundary; S_2 is the constant flux boundary; $H(x, y, z)$ is the head distribution in the simulation area; $H_1(x, y, z)$ is the head at the first boundary; K_n is the hydraulic conductivity normal to the boundary; $\vec{n}$ is the exterior normal direction of the boundary; and $q(x, y, z)$ is the flux per unit area at the constant flux boundary.

3.2 *Seepage parameters*

As it is shown above, seepage depend on water level in the reservoir and on the geology features of the rock matrix. But Kalkani (Kalkani 1992) researched the ambient temperature effect in the Ibitinga concrete dam foundation seepage (Brazil). The investigation concluded the existence a 21 days time lag between the troughs of ambient temperature and the peaks of the piezometer readings. But piezometer measure and seeping flow are interrelated. In conclusion, it is known seepage depends on geology features, water level in the reservoir and ambient temperature.

4 TEST CASE

The test case was La Baells dam. Its main features are:

Localization	Llobregat river, Barcelona (Spain)
Dam typology	arch dam
Maximun height	102 m
Crest length	403 m
Minimum thickness	4 m (crest)
Maximum thickness	20 m (base)
Design flow	650 m^3/s
Reservoir volume	115 hm^3
Drainage basin area	535 km^2

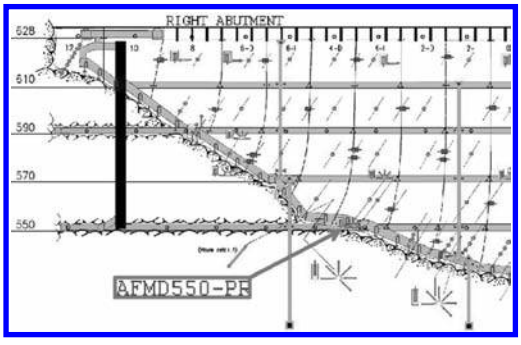

Figure 3. AFMD550-PR flow measurement point of La Baells dam. Source: Catalan Water Agency, 2002.

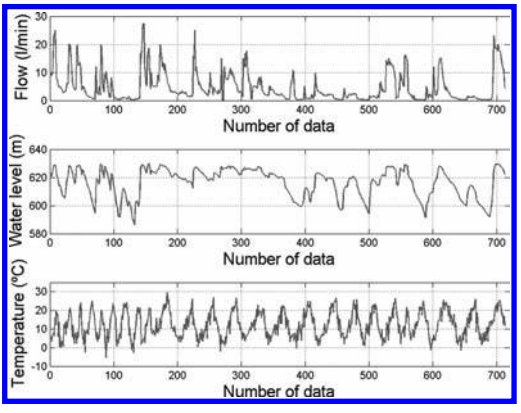

Figure 4. Available data.

The dam site geology is composed of alternating layers of puddingstone and sandstone. Layers are vertical, perpendicular to riverbed. Puddingstone is composed of a mixture of limestone, granite and quartzite, with rounded grains 10 cm average size and its matrix is formed from quadrune.

The dam has 125 drainage holes with 8 flow measurement points (Figure 3) with calibrated containers. The measurement point chosen for this research takes water from the gallery situated in the right abutment at elevation 550 m. This measurement point is called AFMD550-PR.

Data interpretation is now done using a statistical model, which is explained in the next section.

5 STATISTICAL MODEL

A statistical model is used to predict dam behavior, such as uplift in the foundation, displacements or seepage flow. In order to achieve this aim, it seeks relationships between actions and responses. The general expression is given by equation 10.

$$V(t,H,T) = a_1 + \sum a_i f_i(t) + \sum a_j f_j(H) + \sum a_k f_k(t,T) + \sum a_l f_l(t,H) \quad (10)$$

where V is the parameter to predict, t is the chronological time; H is the water level in the reservoir; T is the temperature; a_1 is the independent term; a_i, a_j, a_k and a_l are the coefficients which are calculated using the least squares method; $f_i(t)$ are chronological time-dependent functions; $f_j(H)$ are water level-dependent functions; $f_k(t, T)$ are chronological time and ambient temperature dependent functions and $f_l(t, H)$ are chronological time and water level dependent functions.

A statistical model is developed for the measurement point AFMD550-PR. Its main features are shown in Table 3, where MSE is the mean squared error. There are 1020 available data shown in Figure 4. These data are divided into two sets: the calibration set (608 data) and the validation set (412 data). The adjustment set is used for determining a_i, a_j, a_k and a_l coefficients, and the checkout set is used for assessing the model. These stored data are flow, water level in reservoir and temperature measures.

6 NETWORK TOPOLOGY

The network topology, or network architecture, is the network structure. It is determined by the number of hidden layers and number of neurons in each layer. Most of ANNs applications require one hidden layer, so only one hidden layer was used in this application. The number of neurons in the input layer is equal to the number of input variables. The number of neurons in output layer is equal to the number of output variables. One output variable is desired, flow measured in AFMD550-PR, so only one output neuron is necessary. There is no rule about the optimal number of neuron in the hidden layer. So a heuristic method was applied. It consists on developing several ANN with different number of neurons in the hidden layer and choose the optimal one based on a certain criterion. The proposed criterion was based on checkout MSE. The more validation MSE the network has, the less optimal it is.

Tables 1 and 2 show how the number of hidden neurons influences the checkout MSE for two ANN models. Using the proposed criterion,

Table 1. Number of neurons in hidden layer vs MSE for model I.

Hidden neurons	Validation MSE
8	3.09
9	3.34
10	2.93
11	3.18
12	3.03
14	3.18
16	3.30
18	3.58
20	3.32
22	3.23

Table 2. Number of neurons in hidden layer vs MSE for model II.

Hidden neurons	Validation MSE
7	2.22
10	2.03
15	2.60
20	2.21
23	2.55
25	2.82
27	2.45
31	2.71

the optimal ANN has 10 neurons in the hidden layer.

7 RESULTS AND CONCLUSIONS

Two different ANN models are developed. Model I input variables are the ambient temperature and the water level in the reservoir. However, model II has more input variables. These are ambient temperature, water level in the reservoir, moving temperature averages and moving water level averages of different periods. Ambient temperature periods are 15, 30, 45, 60, 75 and 90 days, and water level periods are 7, 15, 21, 30, and 45 days. As a result, model II has 13 input variables.

In Figure 5, the ANN model I prediction for AFMD550-PR flow and the measured data are shown.

Figure 6 shows the absolute error for the model I, defined as the difference between the measured flow and the estimated by the network. Table 4 shows the MSE and the maximum errors for each set. As a result, model I has a larger MSE than statistical model (Table 3), but the maximum absolute error is very similar to the statistical model error.

Figure 7 shows the ANN model II prediction for AFMD550-PR flow and the measured data.

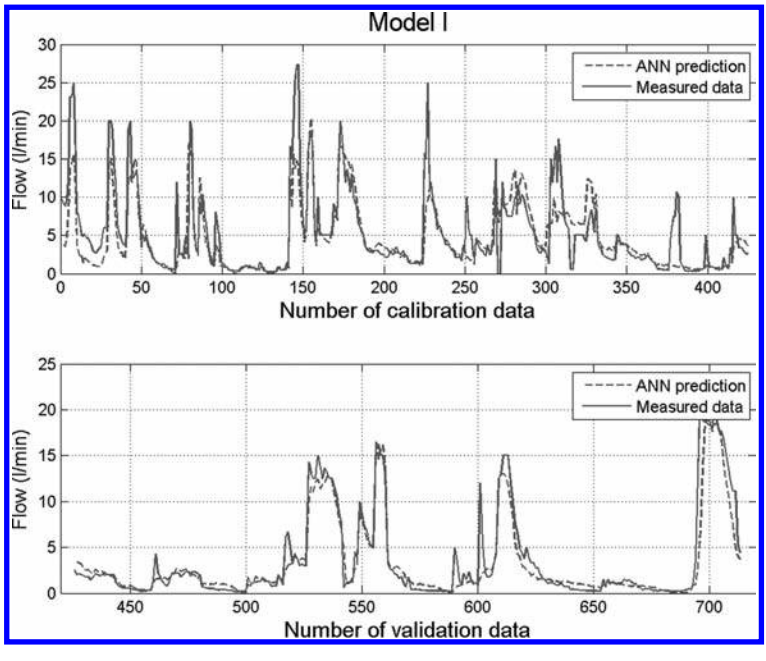

Figure 5. ANN model I prediction vs measured flow.

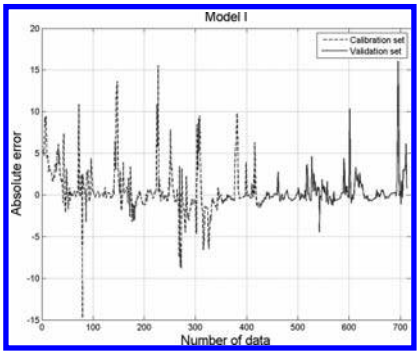

Figure 6. ANN model I absolute error.

Table 3. Main features of statistical model developed for AFMD550-PR.

	Data	MSE	Maximum errors
Adjustment set	608	2.92	15.08/–17.71
Checkout set	412	2.34	10.40/– 8.67

Table 4. Main features of ANN model I developed for AFMD550-PR.

	Number of data	MSE	Maximum absolute errors
Calibration set	425	9.26	15.54/–14.79
Validation set	288	2.93	16.07/– 4.45

Figure 8 shows the absolute error for the model II. Table 5 shows the MSE and the maximum errors for each set. Model II commits, in general, smaller errors than statistical ones.

The advantages of the proposed seepage model are its simplicity to obtain and operate compared with the conventional statistical model. The ANN input variables are the temperature, the water level in the reservoir and moving averages of different periods, but the statistical model is composed of functions and variables which are necessary to hypothesize. ANN does not require to make hypotheses.

Errors associated to the most simple ANN model (model I) have the dame magnitude as statistical ones. When the information provided to the network is increased, as it is made in ANN model II, the ANN prediction is more accurate than the statistical model prediction.

Further research will be performed in order to get more accurate ANN models, determine the variables the phenomenon depends on, develop a methodology to reach the optimal ANN for a variable and future ANN applications to dam monitoring.

This research shows ANNs are powerful tools for forecasting dam seepage. Actions over the dam and its responses are known, but the phenomenon is incompletely known. ANNs are useful to find finding the relationship between actions and responses. This investigation is still in progress

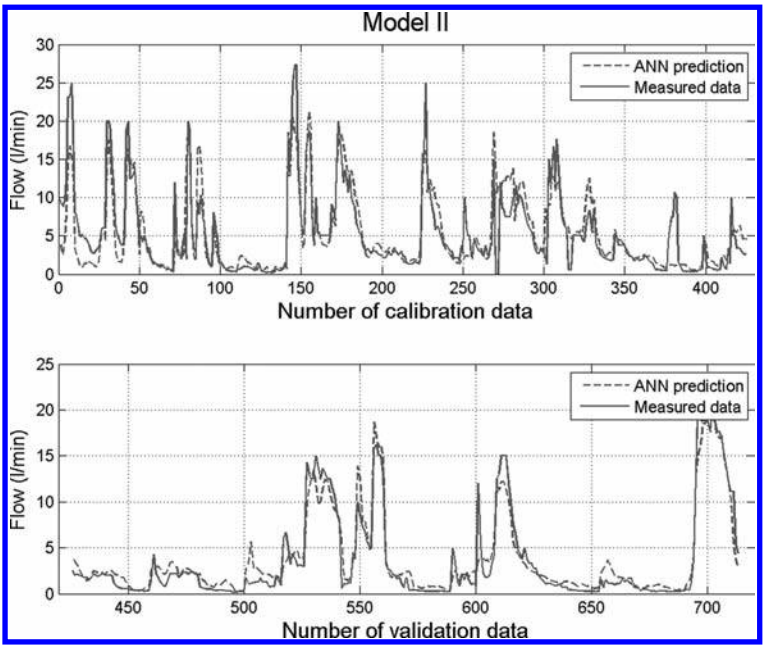

Figure 7. ANN model II prediction vs measured flow.

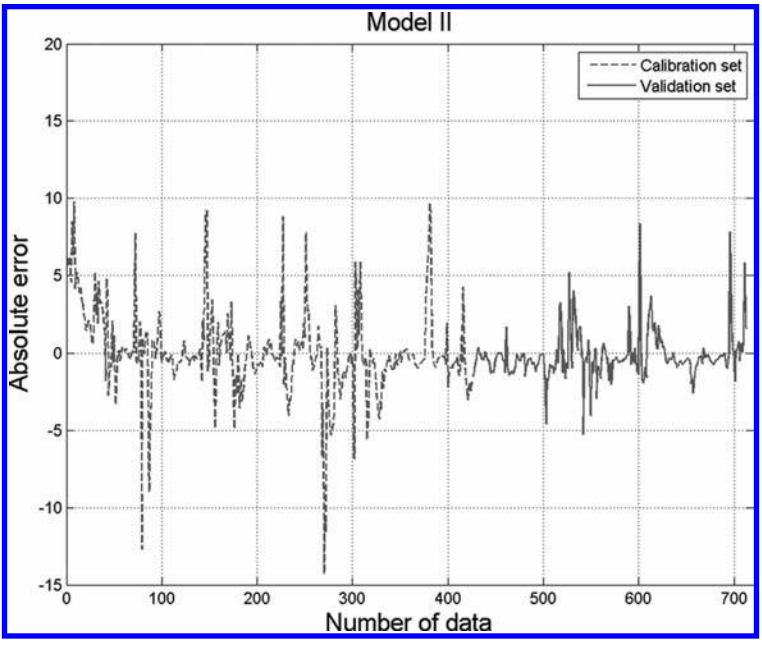

Figure 8. ANN model II absolute error.

Table 5. Main features of ANN model II developed for AFMD550-PR.

	Number of data	MSE	Maximum absolute errors
Calibration set	425	7.67	9.77/-14.33
Validation set	288	2.03	8.36/-5.23

and at this point, the research tries to perform a new methodology for ANN applications to dam monitoring.

ACKNOWLEDGEMENT

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Condition survey of civil structures at Piva HPP

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ABSTRACT: After more than thirty years of exploitation within the project of reconstruction and modernization testing and condition assessment of civil structures was conducted. Performed works included twelve structures of HPP: dam body with galleries and shafts inside, stilling basin, intake structure with valve chambers, headrace tunnels, powerhouse and transformer premises, draft tubes, lower surge tank, tailrace tunnel, grouting galleries and 220 kV switchyard. It pertained to assessment of condition of concrete structures, metal and steel structures and elements, dam monitoring instruments and execution of geological-engineering maps. Specialist services, rendered by climbers and divers were used and dewatering and cleaning of stilling basin was done to perform the inspections and the tests in areas hard to reach.

1 INTRODUCTION

1.1 *Description of the structure*

Piva HPP has been constructed between 1964 and 1976. The plant comprises a dam forming a reservoir, known as Piva Lake, hydraulic head on the dam site, and a power house with a control building, a transformer station and a switchyard. Since the dam site is a narrow canyon, all structures, except the dam, are located underground. The dam is of concrete arch type; cross section and longitudinal section are given in Figure 1. Piva HPP is a storage dam plant with the following main technical characteristics:

- Catchment area 1.757,7 km^2
- Elevation of normal water level in the reservoir 675.0 masl
- Total storage capacity 880 × 106 m^3
- Reservoir length about 40 km
- Arch dam—construction height 220 m
- Rated discharge 3 × 80 m^3/s
- Highest gross head 185.8 m
- Turbines—3 pcs. FRANCIS 166.000 HP
- Generators—3 pcs. three-phase 120 MVA
- Mean annual electrical energy production approx. 744 GWh.

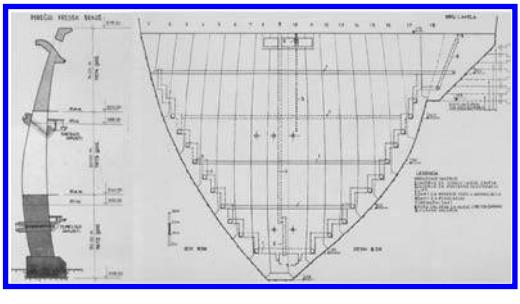

Figure 1. Piva HPP cross section and longitudinal section.

1.2 *Objectives*

The main objective of the tests was to evaluate (and assess) actual conditions of the civil structures within Piva HPP in order to facilitate comprehensive defining of scope of rehabilitation works required to ensure a new, at least 30 years long operation cycle of the equipment and the structures once refurbished. Rehabilitation of Piva HPP is divided in two phases:

- Phase 1: Preparatory Phase
- Phase 2: Execution Phase

The Phase 1 is primarily concerned with execution of tests (condition survey) in order to evaluate (assess) the existing conditions of the civil structures. On the basis of condition survey results, a preliminary design and a feasibility study of rehabilitation shall be prepared. Execution of proposed rehabilitation works shall be implemented in the Phase 2.

The works comprise condition survey of the civil engineering structures consisting of detailed visual inspection, testing of reinforced concrete structures (in situ and using samples), non-destructive testing of pre-stressed anchors providing stability of civil structures and embedded elements of equipment, instruments condition survey, geological mapping and laboratory testing, and dewatering of stilling basin and drainage system condition survey. Piva HPP ground plan is given in Figure 2. Following structures have been encompassed by the condition survey:

1. Dam Body with galleries and shafts within it
2. Stilling Basin
3. Intake Structure
4. Valve Chambers
5. Headrace Tunnels
6. Powerhouse and Transformer Premises
7. Draft Tubes
8. Lower Surge Tank
9. Tailrace Tunnel
10. Grouting Galleries
11. 220 kV Switchyard
12. Appurtenant tunnels and galleries.

1.3 *Conducted works*

All field works have been executed in two stages in period June-October 2009 according to Project Activity Schedule of Work Plan, accepted by the Client prior to commencement of the works. Some works were conditioned by stand still period of HPP Piva, which were rescheduled and shortened.

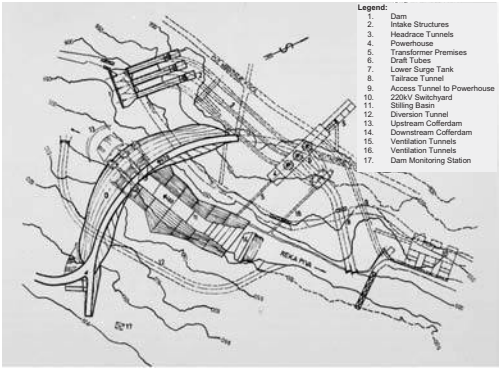

Figure 2. Piva HPP ground plan.

This implicated some changes in applied technology of removing deposit and dewatering of stilling basin.

The Work Plan defined all prerequisites and preventive actions which had to be taken in consideration and implemented in planning, preparation, and performance of the contracted service to ensure that the service performed meets quality related requirements defined in the contract and reference documents. In particular the Work Plan defined organizational structure and responsibilities, activities schedule, inspection plans, inspection procedures, personnel medical skills and physical health protection, equipment, identification, handling and maintenance of inspection records and condition assessment report contents.

In-situ works consisted of visual inspection and in-situ testing of concrete structures and metal works, instruments condition survey, geological mapping and laboratory testing, and dewatering of stilling basin and drainage system condition survey, accordingly to Table 1.

Specialist services, rendered by climbers and divers, were used to perform the inspections and the tests in areas hard to reach. Accordingly, visual inspection of downstream dam body face was

Table 1. List of civil structures and conducted works.

No	Structure	Scope of verification
1.	Dam body	Concrete structure Metal works Instruments condition survey
2.	Stilling basin	Concrete structure Metal works Drainage system Dewatering and removing deposit
3.	Trashrack of the intake structure	Concrete structure Metal works
4.	Valve chambers of intake structure	Concrete structure Metal works
5.	Headrace tunnels	Concrete structure
6.	Powerhouse and transformer premises	Concrete structure Metal works
7.	Draft tubes	Concrete structure
8.	Lower surge tank	Concrete structure Metal works
9.	Tailrace tunnel	Concrete structure
10.	Grouting galleries	Concrete structure Geological mapping
11.	220 kW Switchyard	Concrete structure Metal works
12.	App. access tunnels and ventilation galleries	Concrete structure Geological mapping

Figure 3. Drilling cores (sampling) on the left wall of the stilling basin.

Figure 4. Remote Operating Vehicle (ROV) for underwater visual inspection of upstream dam body face.

performed by climbers (persons trained for abseil works) who were in constant wireless audio and video link with the engineer located on the ground near dam body. Apart from the video recording, the imaging results were simultaneously marked in drawings in an electronic form in order to finally create a map of the damage. Climbers were also used for taking concrete samples from the stilling basin walls as well as for prestressed anchor testing.

Divers were used for visual inspection of upstream dam body face, but only for first 30.0 m. Upstream dam body face below depth of 30.0 m was inspected by a Remote Operated Vehicle (ROV) equipped with a camera and lighting. It was steered from a raft on the surface of the water. A monitor was used to monitor motion of the ROV (shown in Figure 4) and the imaging of the upstream dam body surface. A video record was made simultaneously. In addition to the imaging, depth of the device and actual time of imaging were registered. In a similar way, visual inspection of underwater parts was performed by divers. Besides part of upstream dam body face, divers were used for visual inspection of underwater parts of tailrace tunnel and lower surge tank.

In order to provide access to underwater parts of stilling basin structure, it was necessary to dewater it and clear sediments, which are procedures feasible during total stand still of the power plant only. Condition of the drainage system was investigated separately.

2 CONDITION SURVEY

2.1 *Concrete structures*

A series of records, sketches, drawings, photographs and videos have been made during the inspection. Series of drawings indicate damage by category and photos and/or video recordings as well as in-situ or laboratory test reports.

Types of typical damage are provided in the list below:

1. Crack
2. Crack filled with calcification products
3. Calcification products, spot
4. Segregation
5. Erosion
6. Delamination
7. Scaling
8. Repaired areas
9. Reinforcement corrosion
10. Efflorescence
11. Seepage (line, spot, surface)
12. Wet areas (due to running water flowing from surrounding surfaces/joint; due to condensation)
13. Organic pollutants (moss & moulds).

Starting with dam body as the most impressive structure of Piva HPP and structure which requires special approach, the main problem pertains to numerous occurrences of seepage through the dam body, varying in intensity and location. Since beginning of use of the dam, measures aimed at stopping the seepage were implemented repeatedly, with more or less positive results. Seepages through the dam body observed do not pose a threat to mechanical resistance and stability of the dam, as those wash away weak calcium hydroxide component of the concrete, which contributes little towards that resistance.

In the past, condition of concrete of the dam body was affected by a number of factors, such as construction method, climate and structure-tensile properties. The downstream surface of the dam (Figure 5) has constantly been exposed to climate changes, seasonal and daily, as well as to

atmospheric influences. Therefore the damage, which is a consequence of imperfections related to execution of construction works, cold joints and working joints, is of a greater degree in some places. Somewhat more intensive weathering is observed in zones of segregation of concrete, while the working joints and a number of cracks represent places of ingress of water through the dam body (see Figure 6). At downstream dam body face, slope diagonal cracks were observed in the lowest third of dam body height. Since inspection was done in the beginning of summer, these joints and cracks were closed due to external temperatures, and the leakage was of a lower intensity, or absent altogether. In order to achieve at least a referential depth of sloping cracks, ultrasonic measurements were performed as shown in Figure 7, and those revealed that the cracks propagate up to 1.0 m deep.

The upstream face of the dam exhibits no visible damage, apart from localized segregation along horizontal working joints and along cold joints between lamellas, no different from the downstream face of the dam.

Figure 5. Downstream dam body face.

Figure 6. Visual inspection of upstream dam body face; water leakage at horizontal working joint.

Figure 7. Crack ultrasonic measurement at downstream dam body face by climbers.

On the inside of the dam seepage and ingress of moisture are pronounced at cold joints and at horizontal working joints, mostly on the upstream side closer to abutments of the dam.

Spot repairs were carried out on a number of occasions where seepages, superficial segregation and corrosion of the reinforcement steel occurred, by means of grouting or application of thin layers of mortar and coatings. The grouting has stopped ingress of water generally successfully, however in places there is still some seeping/ingress of moisture and calcified sediment is being deposited.

Superficially rehabilitated areas in the main revision horizontal galleries, and elsewhere, are characterized by lack of proper bond between the thin layer of mortar/coating and substrate (when tapped on, it gives off a hollow sound) thereby concealing moist spots and/or spalling concrete accompanied by reinforcement steel corrosion, which appears to be mostly localized and limited to bars with insufficient protective layer of concrete.

Proposed rehabilitation measures for dam body include grouting of the cold joints (vertical joints between lamellae), working joints and existing cracks at the downstream face and rehabilitation of damaged, segregated and eroded concrete with high quality remediation mortar; the same rehabilitation works should be performed on the upstream face throughout the zone of regular fluctuations of the water level. Thence, for the interior parts it was proposed to repair all worn and lose parts with water impermeable mortar as well as reinforcement cleaning and protection, and remediation of cracks and the working joints for preventing seepage through the concrete.

Other structures under consideration are, as far as concrete is concerned, in a relatively good condition, expected for age of the structures and the environment in which those are situated. As the structures are mainly underground, apart from damage occurring as a consequence of imperfections of execution (poorly executed working joints, segregations, inadequate protective layer of concrete), further damage occurred during their use, consisting of seepage with efflorescence and washed off cement matrix and carbonation, as is characteristic in the powerhouse and intake structure valve chambers. In order to achieve an extension of service life as well as usability in terms of ensuring operating and safety conditions required, it is necessary to undertake certain measures regarding protection and repairs.

In order to provide appropriate conditions for inspections and tests of stilling basin, dewatering and removal of deposit contained therein was performed. Visual inspection, in-situ testing and core samples taking were performed at different levels during different periods, according to water level decreasing. The damage observed: spalling and fragmentation of surface layer of concrete as a consequence of freeze-thaw cycles and up to 2 mm wide cracks on both sides of the top of the stilling basin, vertical cracks in walls—caused by shrinkage of concrete partially running through open working joints. The cracks generally exhibit larger gaps (estimated up to 5 mm); water originating from bedrock seeps through individual cracks; horizontal cracks, up to several metres long, generally arranged along or within working joints, with efflorescence and filled in; horizontal working joints with traces of efflorescence, generally dry and some moist at the time of the inspection; localized segregation, generally along the horizontal working joints, up to 10 cm deep; exposed reinforcing steel, spalling of concrete in places of inadequate protective layer of concrete—those areas are localized, mainly above the horizontal working joints; spalling of concrete is present along edges of stilling basin stairs; erosion of surface of concrete of the stilling basin base slab.

It was concluded that the shortcomings determined do not affect stability and carrying capacity of the structure, nor its service life. Some minor repairs are recommended.

Powerhouse and transformer premises, as well as lower surge tank, valve chambers of intake structure and 220 kW Switchyard are situated underground. Damage observed in these structures are in accordance with their placement: typical ingress of moisture or leakage along the working joints and the cracks (occurred due to shrinkage) with visible traces of long term seepage as efflorescence or incrustations; corrosion of the reinforcement steel was observed in specific areas, where the concrete protective layer is insufficient and segregation of the concrete typically occurred next to the working joints.

On the basis of an excellent correlation of the compressive strengths and the core densities, it was concluded that principal cause for a poor and uneven concrete quality is lack of care in respect of addition of water during its production, and probably during pouring of the concrete as well. For example, designed class of the concrete, according to technical description in the design, is MB22 for all elements of the powerhouse. Results of the compressive strength tests performed on the cores of PTP are considerably uneven, ranging between 23.5 and 56.8 MPa (Figure 8).

Higher values of carbonation were observed in Powerhouse and Transformer Premises (PTP), see Figure 9. It was assumed that observed carbonation might be, or that it already is a cause of corrosion of the reinforcement steel (since the designed protective layer of concrete is 3 cm thick). During the carbonation process carbon dioxide penetrates the concrete and reacts with calcium hydroxide forming carbonates. That reaction decreases pH value of the concrete below 9, thereby initiating process of corrosion of the reinforcement steel. Since such process is determined in this case in level of the reinforcement steel, and conditions in which the structure is found are "the most favourable" for carbonation of the concrete (in terms of temperature, and especially in terms of humidity ranging between 40% and 70%), it was proposed

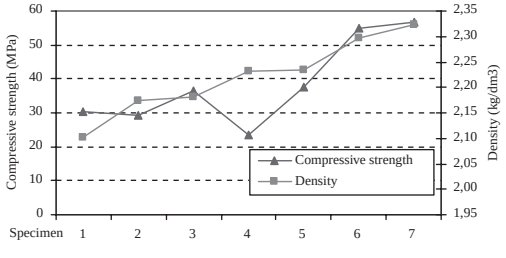

Figure 8. Correlation of compressive strength and density in powerhouse and transformer premises.

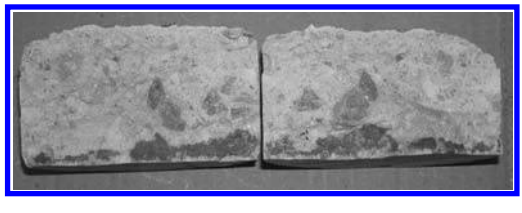

Figure 9. Determined carbonation depth varies between 27 and 48 mm.

to remove carbonated concrete layers and repair all the reinforcement steel exhibiting (visible) corrosion and to protect all surfaces with migration inhibitors and coatings against carbonation.

Headrace tunnels were inspected immediately after they were emptied. There was accumulated silt, fine, powdery, wet material, on entire surface of the concrete. Working joints, i.e. places where continuations had been made in the lining, are visible. At these places, there are segregations, as well as water leakage from the surrounding bedrock. Appearance of cracks was also noted at the top of tunnel, along the tunnel axis, as well as a certain number of oblique and vertical cracks on lateral walls. These kinds of deteriorations are also noticed in tailrace tunnel, grouting galleries, access tunnels of grouting galleries and service chamber. Furthermore, in headrace tunnels in some places, concrete was subsequently repaired using mortar. Those repairs were most probably made immediately after removal of formwork during construction. Due to water flow, loss of adhesion between mortar and concrete is observed.

Rehabilitation measures for headrace tunnels provide cleaning of entire exposed surface of the concrete; cleaning of all areas with any subsequently applied mortar or concrete, as well as segregation zones and any loose segments; repairing of all thus prepared surfaces using high-quality mortar and concrete for rehabilitations, with high performance in terms of adhesion to the old concrete; cleaning and protection of corroded reinforcement steel; cracks grouting using epoxy resin based grouts.

All these rehabilitation measures have been proposed for other structures like grouting galleries and access tunnels, as well as draft tubes.

In draft tubes a layer of 3–5 mm of degradation surface was observed. Surface of concrete is slightly eroded and exhibits decreased hardness when tested with Schmidt hammer. This was determined in all areas tested by Schmidt hammer (10 tests performed in each draft tube), therefore allowing an assumption that all surfaces are in that condition. Other damage observed are minor and the same as above mentioned.

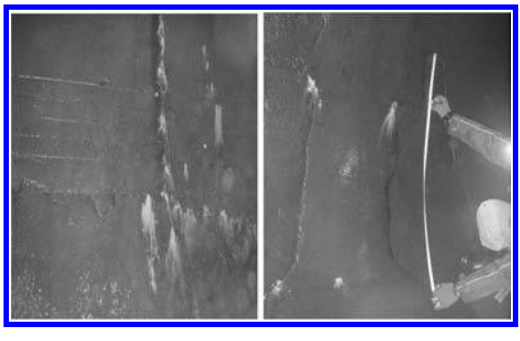

Figure 10. Calcification products at working joints and loss of concrete-mortar adhesion.

Table 2. Overview of compressive strength results.

No.	Setting	Designed C.S. class MPa	Characteristic in-situ C.S. MPa
1.	Dam body	30	47.7*
2.	Stilling basin	30	34.7**
3.	Trashrack of the intake structure	–	–
4.	Valve chambers of intake structure	22	33.4
5.	Headrace tunnels	22	21.2***
6.	Powerhouse and transformer premises	22	27.5
7.	Draft tubes	22	40.2
8.	Lower surge tank	22	45.0
9.	Tailrace tunnel	22	38.4
10.	Grouting galleries	30	34.5
11.	220 kW Switchyard	22	25.7
12.	App. access tunnels and ventilation galleries	22	25.5

* Cores taken underwater
** Correlation between cores and Schmidt hammer
*** By Schmidt hammer testing.

Investigation works included in-situ testing of concrete strength with Schmidt hammer, ultrasonic measurement of crack width, taking core samples from structures for concrete strength assessment and carbonation depth measuring.

Table 2 provides an overview of compressive strength (C.S.) results with relations of estimated and designed compressive strength.

2.2 *Dewatering and removing deposite and Drainage system*

During the second part of the works, while entire plant operation was suspended, the stilling basin was dewatered and cleaned, and condition of the drainage system was assessed.

Approximately 4.700 m^3 of material, bulk deposits and silt has been removed. The deposit varied in composition, ranging from construction debris (remains of mortar, concrete, gravel, anchors, wire cables etc.) to material subsequently transported into the stilling basin, mostly in form of material falling down canyon slopes (rocks, trees, branches etc.).

During that operation the survey of condition of stilling basin drainage system was executed. As far as the drainage system is concerned, it has been determined that access to drainage galleries or

Figure 11. Stilling basin of Piva HPP.

Figure 12. Emptied stilling basin.

drainage channels is not possible. An access tunnel to the drainage system was checked by divers. It was determined that pump installed in pumping station of the drainage well cannot dewater the stilling basin because of clogging, as assumed in tender documentation. This link between the pumping station and the stilling basin was restored using compressed air fed through a tube and clearing of deposits from the drainage pit. In spite of that, final 40 cm of water and silt could not be drained since it was not possible to establish automatic operation of the stationary pump, as well as due to shortened period allocated for performance of the works which effectively prevented resorting to any other option. Therefore the bottom slab of the stilling basin could not be visually inspected, and only test cylinders were drilled and retrieved for testing and an inspection of their upper surface.

2.3 *Metal works*

A visual inspection and metal structure testing encompassed the following part of structures:

- Hook load testing with applied tension force of 125 kN. Full intensity was reached in five equal phases, (Valve chambers of intake structures);
- The thickness of the steel elements was checked in several places using an ultrasonic instrument in order to determine depth of corrosion partially affecting these profiles, (Lower surge tank);
- Control measurements of diameter of the bars used to manufacture the hooks, (Lower surge tank);
- Load testing applied to vertical girders for insulator, (Switchyard);
- Load testing applied to cantilever girders, (Switchyard);
- Load testing applied to truss girders, (Switchyard);
- At anchors, where possible, individual wires of the anchor were pulled using specially designed hydraulic equipment. This is performed only if length of the wires is sufficient to be attached to the press in front of the anchor end block. Thus the wires may be tested against failure using a programme for testing a given test load (the load is determined as 90% of the expected load calculated at the time of the test, and amounts to 30–35 kN per wire Ø 7 mm). When a cable fails before preset test load is achieved, the load causing individual wires to fail shall be measured, (Switchyard-Exit portals, Stilling basin);

Relevant VSL and BBRV publications were also used as additional documents important for preparation of the testing programme and the testing itself.

By means of a detailed inspection of the site it was determined that the anchor is of BBRV type. It consists of 12 Ø 7 mm wires. 6 (six) of the wires took protrude out of the anchor plate. Their exposed lengths are about 1.0 m long. Accordingly, two of the wires were pulled a using specially made, type HP 40, hydraulic press (maximum allowed load 40 kN, length 150 mm, working step 120 mm). The press was calibrated using a manual, type PU 30, hydraulic pump. A calibration diagram indicates relation p (bar)—F (kN) as $P = 9.47 \times F - 10.5$; 5 kN £ F £ 35 kN; p—rounded to 5 bar.

Displacement of the first wire was achieved under pressure of 90 bar (10.6 kN). Immediately after the displacement the pressure fell to values in 50 bar—30 bar range. That wire was fully extracted from the anchor. Location of wire failure was determined at 3.55 m from the beginning of the anchor plate face. The failure exhibits no contraction.

The data contained in the tender documentation specifies that 316 pre-stressed BBRV type anchors are installed, with bearing capacity ranging from 240 kN to 670 kN whose length

are 8, 10, 15 and 20 m, while the anchor plates are "impressed into the concrete" (heads of the anchors are located in square niches). The anchors are installed in the vertical walls and on the bottom of the stilling basin.

The inspection of the stilling basin vertical walls discovered failure/partial failure of one anchor.

2.4 *Instruments*

Inspection of dam body monitoring instrumentation included inverted plumblines, Rittmeyer water balance installation and Interfels distometers in order to reveal cause of functionality and serviceability problems of these instruments.

In order to inspect position of the inverted plumbline strands within the tubes and condition of tube surface, possible objects or presence of water, it was necessary to use video camera which facilitated quantitative determination of position of possible irregularities along the vertical direction of the tubes.

A qualitative inspection of Rittmeyer installation (Figure 14) in the measuring premises has revealed that the equipment is not functioning. Severe damage to the equipment has been observed, due to its idle period. According to the information of the technical service, at the moment, the reservoir level is measured using a new pressure probe installed on the face of the dam.

Plausibility of future usage of the deflectometers with regards to their condition and possibility of connection to a future data acquisition system was reviewed.

Built-in deflectometers manufactured by Interfels are chain deflectometers placed to detect

Figure 13. Video inspection of reverse plumbline.

Figure 14. Rittmeyer equipment used to measure water level, is entirely unusable and out of order.

deflections within rock in the horizontal plane perpendicular to the vertical plane which contains tangent on the middle line of the dam arc on the corresponding elevation. Each deflectometer probe is mounted into casing which is placed in separate borehole, and consists of heads and linked together by distance rods. Head contains sensor for displacements measurement which, due to the geometry of the sensor, provides change of the angle resulting from displacement of the corresponding rod that is wire inside the rod.

Works on deflectometers testing included: checking of electronic components in the substations, checking of integrity of connectors, cleaning for approach to the contact heads, checking of wiring between substations and contact heads and testing of sensors in each measurement segment by measurement bridge simulator.

Dam monitoring instrumentation, inverted plumblines, Rittmeyer water balance installation and Interfels distometers are generally nonfunctioning and in a poor condition. Besides, technology employed by similar instrumentation is at a substantially greater level today, offering more acceptable, better solutions and more options.

Efforts should be made to establish an automatic and centralized system for monitoring and processing of the measurements.

2.5 *Geological mapping*

Geological mapping of unlined tunnels and galleries encompassed verification of condition of excavated rock after 35 years in order to determine specific zones requiring special action in order to ensure their stability.

Engineering—geological mapping was executed after tunnel excavation. Mapping included detection and marking of all general geological and structural geological formations, but primarily engineering-geological

and hydro-geological formations significant for evaluation of a rock mass quality.

Engineering-geological parameters which are significant for the geo-mechanical classification of rock mass were especially monitored according to Bieniawski 89. Discontinuities were marked (bedding, faults, joints), their position and orientation, as well as length, aperture, width, stability and type of infilling. Moreover, karst formations are marked, like caverns, caves, pits, canals etc. Mapping is performed through continuous monitoring, surveying and charting of geological formations in the crown as well as in the left and right walls. All noted elements are charted into a developed longitudinal engineering-geological cross section of the tunnel in scale M 1:100.

Recommendations have been made in respect of remediation of caverns and individual unstable zones or localized instabilities.

3 CONCLUSION

Generally, basic input data for assessment of any remediation measures necessary are assessed condition of the structure with defined categories of damage and service life desired, in terms of both the repair itself and the structure as a whole.

Tender document states as follows: "The main objective of the tests is to evaluate (and assess) the actual conditions of the civil structures within HPP Piva in order to be able to fully define the scope of rehabilitation works required to ensure a new, at least 30 years long operation cycle of the equipment and structures refurbished".

However, when considering the structure or elements securing the structure (like pre-stressed anchors), it is necessary to adopt a different approach. Considering its present age of somewhat more than 30 years, taken together with required additional 30 years, that would comprise approximately 70 years, which certainly cannot represent a total required service life of a structure of this type and level of importance.

When one considers the design task in a strict sense, it could be said that the structure is currently in such a condition that it is possible to foresee its operability during the following 30 years as well, without implementation of any significant remediation measures. On the basis of conditions determined during this inspection, it may be assessed that the structure would sustain all effects of regular use and all environmental effects in that period, with regular maintenance and minor repairs, and that the foreseeable effects would not cause failure, deformations beyond permitted levels and consequent damage to structural elements or equipment and disproportionally great damage to the structure relative to causes of their occurrence. In that context, as the damage entailed lies within range permitted, proposals regarding remediation works contained herein would most definitely ensure the service life of the structure as requested.

On the other hand, Eurocode 1991-1 requires designed service life for structures of this type of at least 100 years, and the fact is compounded by importance of HPP Piva as the uppermost dam within a system of electric power plants in Drina River catchment. Therefore, when determining approach and scope of any remediation, a significantly longer required service life than the one requested need be kept in mind.

In respect of equipment, the request is appropriate both in terms of dam monitoring instrumentation and metal equipment such as lifting hooks, fixing anchors, embedded parts of equipment supports, trashracks or bars.

The visual inspections have revealed that most of the structures were poorly or inadequately maintained in the past. This primarily pertains to drainage of seeping water as well as drainage of technological water.

Finally, particular attention need be paid to the maintenance. Operating and maintenance programme should be based on regular inspections of the structure which should be defined in terms of their frequency and level. Monitoring and review of results of the inspections performed over a specific period provides a basis for decision making regarding the intervention and facilitates planning and optimization of expenses. The results of the inspections comprised by the works related to this project may represent a "zero" condition and frequency of inspections of similar scope should be set at a 10-year basis.

In order to maintain an appropriate level of usability and safety of HPP Piva structures during its designed service life it is recommended that a structure management system be established and implemented along with regular maintenance, and minimum investments. Such a system entails collecting and processing information (technical data, list of archived documents, information on inspections and interventions performed), as a database used as a basis for analyses which are to be performed in order to determine future needs, project future conditions, analyze alternative designs and develop programmes and plans.

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Conservation and maintenance

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Operation and management of Karun 4 dam

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ABSTRACT: The 1000 MW Karun 4 Hydropower station, located in southwest of Shahr-e-Kord city in Chahar Mahal and Bakhtiari province, is one of the largest hydropower projects in Iran.

Partly financed by the Iran's Power Ministry, the project is being built and operated by Jahad Tosse Manabe Ab Company.

Karun 4 dam is a 230 m-high double curvature concrete arch dam, the highest dam in Iran and the fifth highest of this type in the world. Concreting of the dam started in 2000, reservoir will be filled 2010, and dam construction will be completed in 2009. The dam has been in safe operation for nearly 10 years.

This paper gives a full description of the dam operation, and presents a complete summary of the management experiences achieved and management modes established in the aspects of dam safety, dispatch, technology and environment.

Karun 4 dam operation and management can be of some reference for other similar dams both in Iran and in the world.

1 INTRODUCTION

Originating from Zagros Mountains, Karun is the largest river in Iran, which enters Gotvand and Khuzestan plain in southwest of Iran at the downstream and ultimately joins the Persian Gulf.

In recent decades, large strides have been employed to exploit the hydroelectric energy in consideration of the ever-increasing demand for energy in Iran. Namely, construction of a series of Karun dams, at the kilometers 377 to 747, was started in order to control the floodwaters of the river and to generate the required hydroelectric energy.

1.1 *History & Records*

The Primary studies of Karun 4 project were conducted within the framework of water resources development project, as well as the overall planning of water resources management of Karun river basin in 1966.

The first two stages of the studies were performed during the operation phase of the project.

The executive operation of this project was launched in 2002. Upon completion of the diversion system operation in 2004, the concreting operation of the dam body was begun right at the same year and it was filled by 2009.

2 PROJECT SUMMARY

Karun 4 dam reservoir is located at 180 kilometers southwest of Shahr-e kord city, in the Chaharmahal & Bakhtiari Province, and at the upstream; in the confluence of the two Karun & Monj Rivers. The height of this dam is 230 meters from the foundation level and accordingly this dam has earned the title of "The largest double-arch concrete dam in Iran". The Capacity of the power plant is 1000 MV, which generates 2107 GV-hour electric power, having four turbines.

The objectives of this project were; to regulate the water discharge of Karun River to 3.7 billion m³ per year, to control the destructive floodwaters of this river and generating hydroelectric energy.

The dam consists of a double-arch body with 230 m. height and has a 440 m. crest. The crest is 7 m. in width and there is a spillway structure with two types of free (5 parts) and segmented gates (3 parts) with the capacity of 6150 m³ per second. The power plant has the capacity of 1000 MV

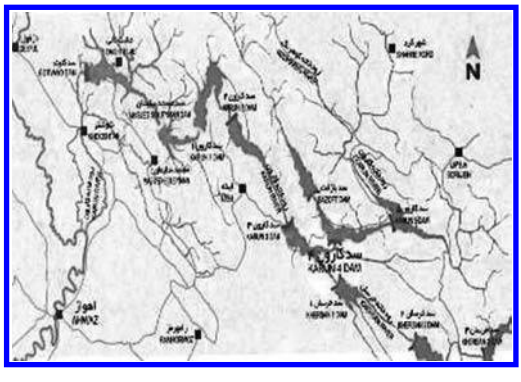

Figure 1. Location of the dam.

(4 turbines of 250 MV), and a diversion system with cofferdam in 45 m height (stair) and a tunnel with 9.5 m. in diameter (2 parts) and the total length of 1355 m.

The total volume of reservoir is 2190 million m³ and its surface area is 29 km² and the lake length in Karun and Monj branches is respectively 41 and 28 kilometers.

Flood discharge of Karun River in dam 4 site is 6,155 m³ per second for 1000-year floods. Water tunnels of the power plant include 4 tunnels with total length of 1,401 m. and perpendicular shafts of 212 m. high.

2.1 *The dam executive operation*

The Executive operation of the dam was launched from 2002 and faced with numerous complicated engineering situations.

In 2004, while the foundation of the dam was under construction and the drilling was performed at the flanks, the river surged as a result of a three-day rainfall. Hence, the river discharge reached 5,500 m³ per second and water inundated the cofferdam and entered the operation site. Having over passed the cofferdam for 40 minutes, this structure was shattered on the right flank and the project suffered heavy damages.

In an another development project in 2005, the inter-discharge of Karun River reached 5750 m³ per second and the cofferdam was destroyed for the second time and caused 7 months delay for the executive operation.

Despite all this, the project crew was still determined to run the filling phase prior to March 20, 2010 with non-stop efforts on following the operation schedule. Within the scope of this project, 2000 personnel are effectively working on dam body which is the highest record achieved in civil projects in Iran. Upon completion of the deviation system in 2003, executive operations of dam foundation and the dredging were embarked with a special technique. Water discharge pumps were employed to remove the surface waters and mud from dam foundation.

Owing to the high volume of mud and water, most of the pumps did not attain the required efficiency and were wrecked soon. So the project contractor presented a special plan which was confirmed by the Consulting Engineers. The plan was to mount all pumps on a vessel and the discharge was performed from the surface by collecting all the water from the surface of the foundation and dredging the river drifts and mud finally became possible.

2.2 *Mode of concreting the dam body*

Following extensive studies on the mode of concreting, technical group decided to use an aerial cableway

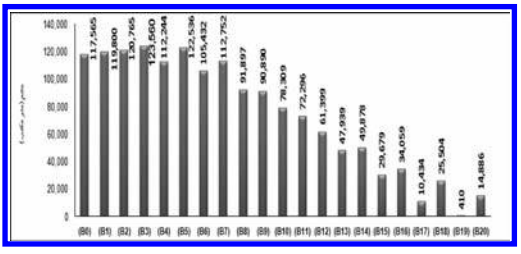

Figure 2. Concrete volume.

and upon securing the client's confirmation; the required components were purchased from abroad.

In the course of excavating the dam in the left and right flanks, using a radial aerial cableway and by constructing an arc-shaped runway in the left flank and a fixed point on the right flank.

Two aerial cranes were used for concreting the body each with the capacity of 28 tons and load handling capability up to the height of 290 m.

A river crusher and a mountain crusher were used for the production of concrete aggregate and two batching plants were employed for concrete production with the capacity of 120 and 240 m³ per hour.

The dam body concreting was embarked from 2006 with a total concreting volume of 1.6 million m³.

Given the remarkable assets of the aerial cableway, taking the benefit of employing the full capacity of concrete production plant and highly experienced and devoted personnel, the contractor managed to register a concreting load of 89,000 m³ per month in 2008, which hits the records of concreting in the Iranian dam-building history.

3 DAM SAFETY MANAGEMENT

(Organizational systems and regulations for safety management of Karun 4 Dam).

Karun 4 is the third power station built on the Karun River. The Internal HSE Unit conducts the dam safety management at two levels, i.e. the company level and the power plant level. The management organization is structured such that there are neither uncovered areas nor repeated functions. A clear reporting line is established, so that everyone in the organization is clear for what and to whom he is responsible.

The deputy general manager in charge of power production is responsible for dam safety management, coordinating and solving major issues in dam safety management. The Production Management Department is the main functional department involved in dam safety management, responsible for inspections, directions, supervision and performance appraisal.

Figure 3. Upstream view.

Dam safety management is led by the director of HSE Unit, assisted by the deputy director (who is responsible for power production) in the area of dam operation, and by the chief engineer in the area of dam safety supervision and related technical matters. The civil works department of the power plant is the executive and responsible department for dam safety, supported by one civil works maintenance group and one civil works observation group, responsible for routine patrol inspection, safety monitoring, and maintenance and repairs.

Dam safety management is governed by rigid laws and regulations. Pursuant to the pertinent Iranian laws, and based on the actual circumstances, HSE Unit formulated a series of rules and regulations, such as Regulations on Dam Safety Management, Flood Control Management Rules, Reservoir Operation Management Rules, Rules on Management of Hydraulic Works, Approaches to Dam Safety Supervision and Appraisal, Specifications for Civil Works' Observation, Regulations for Maintenance and Repair of Hydraulic Structures, Regulations for Reservoir Dispatch, Regulations for Operation of Spillway Gates, Supervision Rules for Hydraulic Structures, Patrol Inspection Rules for Hydraulic Structures, Management Rules for Dam Safety under Flood Control Circumstances, Rules for Management of Flood Control Materials, Organizations and Responsibilities for Flood Control and Dam Safety Inspection, etc.. Establishment and implementation of these rules and regulations have clearly defined the responsibilities, standardized the behaviors, and encompassed dam safety management under these laws and regulations.

4 RESERVOIR OPERATION MANAGEMENT

Karun 4 reservoir does not obey seasonal regulations, so a flood control task has been developed in the design. To be a station by the power generation, as its main purpose, a flood control water level or flood control storage has been designed. The normal level was consistent with the designed flood level. The spillway structures have a high discharge capacity. At the normal level, floods of 1000-year probability can be fully discharged (including the normal generation flows).

5 DISPATCH PRINCIPLES FOR POWER GENERATION AND FLOOD CONTROL

Power generation of Karun 4 follows the dispatch orders of IWPCO (the owner), and the guidelines for season floods dispatch try to keep "the balance" between the inflow and outflow, considering the following points:

- Dam "safety first"
- The maximum outflow is not higher than the flood peak at the same time;
- Power generation is duly considered.

Iran's Hydrological Information Center is in charge of the flood dispatch of Karun 4 reservoir. The gate project for flood control was ordered by the hydrological information center and operated by Karun 4 Power Plant.

6 ORGANIZATIONAL STRUCTURE FOR FLOOD CONTROL

Flood control is not only related to the safety of the station, but also to the life and property safety of the people in both upstream and downstream of the station, so it is a critical issue in dam safety management. Under the principle that the chief executive officer is held responsible for the flood control, the contractor (JTMA) has set up a Flood Control Leadership Group headed by the general manager, fully responsible for decision-making on flood control works of all company projects. The deputy leader of the group is the deputy general manager in charge of power production, and the group members are the other deputy general managers. The Production Management Department is the executive organization of the leadership group, ensuring that the pertinent laws and regulations are properly established, approving the flood control plans and flood dispatch issues of the power station, organizing flood control inspections, and reviewing treatment plans for disastrous hazards or incidents.

7 TECHNICAL MANAGEMENT OF KARUN 4 DAM

Technical management is an important support and assurance of dam operation and safety. Technical management (Chief Engineer) of Karun 4 dam is in charge of the IWPCO. The Deputy Chief

Engineer for civil works (JTMA, as a contractor) was responsible for specific technical works related to dam operation. Down the line, there are working groups covering different professional areas, each having a Supervisor Engineer and a Maintenance Engineer, specifically responsible for observations and technical maintenance. Since its operation, a number of technical innovations and research work have been carried out, giving a strong support to safe operation of the dam.

8 MODIFICATION OF SEEPAGE MONITORING SYSTEM

On Karun 4 project, 44 piezometer orifices are established around the dam, most of which are in the slopes on both sides of the plunge pool. Affected by spillway sprays, measurement during the flood season is often impossible, resulting in a gap in the observation data and thus influencing the objectives and the accuracy of the evaluation of the seepage field of the dam. In 2006, an automation project was utilized to put the 31 orifices, with respect to the observation difficulties during the wet seasons under the coverage of the Measuring and Control Unit (MCU). After this modification, all instruments have been performing well, ensuring continuity and completeness of the observation data.

9 MODIFICATION OF EARTHQUAKE MONITORING SYSTEM

Karun 4 dam is built in a quake-prone zone. The earthquake intensity at the dam site is 7 degrees, and the design criterion is 9.2 degrees. To monitor the impact of earthquakes on the dam, an earthquake monitoring system has been set up. Previously, there were 10 seismic detectors, but it was later found out that they failed to record some of the earthquakes occurred in the adjacent areas. Analyses indicated that this had much to do with the devices and their layouts. In 2006, adjustments were made to the system, with new models of devices installed in a more reliable layout and the total number of earthquake detectors increased up to 24 pieces. In subsequent operation, the system accurately recorded a number of earthquakes occurred in the surrounding areas, providing reliable data for analysis of earthquake impact on the dam.

10 ENVIRONMENT MANAGEMENT

Assessment of environmental impact of Karun 4 Project started in 1995. Investigation and assessment were carried out over ten topics, which included local climate, landslides, soil, terrestrial life, aquatic life, historical relics, etc. During the construction and operation period, close attention was paid to the environment management in order to reach the environment-friendly goals both in the construction period and long-term operation of the project.

11 ENVIRONMENT PROTECTION DURING CONSTRUCTION

Environmental work during the construction period mainly involved disposal of construction spoils, treatment of waste water, dust and noise control, and health protection of the construction staff. Major environmental measures were among the following:

- Construction spoils were disposed at designated areas, which were properly protected;
- Construction areas were landscaped, with a total landscaping area of 62.7 hm^2;
- Treatment facilities were built to treat waste water from the concrete batching plants;
- Wet-production method was used in aggregate crushing plants;
- Hospital and first-aid stations were established on the project site;
- Sewage treatment plants were provided in the construction sites and living areas;
- Low-exhaust and low-noise machinery were used to reduce exhaust emission and noise;
- Ear plugs and ear muffles were provided for each member of the construction crews for personal health protection.

12 RESERVOIR BANK PROTECTION

The valley slopes at Karun 4 are high and steep, so landslide occurrence probability is rather high. To improve the soil retaining ability of the reservoir banks and the ecology of the reservoir area, a demonstrative forestation buffer zone of 240.3 hm^2 was established, with a survival rate of 90%. Subsequent to the establishment of the demonstrative zone, from 2002 to 2005, more than 1,000 hm^2 of seedlings were planted. Besides tree planting, a number of areas have been defined as special zones, limiting human activities for the purpose of forest protection. The government of Shahr_e_Kord Municipality established the Karun 4 Scenery Administration, which has also contributed to protection of the vegetation around the reservoir.

13 ENVIRONMENT MONITORING

Based on the actual requirements of environment protection, environmental monitoring was carried

out during the construction period, which covered meteorology, water quality, hydrology, terrestrial life, and aquatic life. Some of the monitoring items have been carried out over the operation period. Environment monitoring was not only necessary for understanding of the environmental conditions in the project area, but also provided invaluable data for future projects in pre- and post-construction comparison and analysis of environmental changes.

14 CONCLUSION

The executive operation and management of Karun 4 dam complies with the policy of "safety, efficiency and environment-friendliness". On one hand, all laws and regulations pertinent to safe production, dam management and environment protection are strictly maintained, and on the other hand, exclusive efforts are made to explore operational and management experiences and undertake technological innovations. The management experience in safety, dispatch, technical aspects and environmental aspects accumulated over the years will provide a good reference for construction and operation of the company's subsequent projects, and may be of certain reference for other similar dam projects in Iran or even in the world. Meantime, JTMA will pay close attention to the latest developments in dam management nation-wide and world-wide, and learn advanced experiences and technologies from other projects, so that we can keep improving our management level to ensure project safety and provide better social and economical benefits that Karun 4 Power Station can offer.

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Trabajos de mantenimiento y conservación en el sistema de presas del Ambroz (Cáceres, España)

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RESUMEN: La presente comunicación tiene por objeto analizar diferentes aspectos relativos al mantenimiento y conservación de presas a partir de la experiencia en el sistema de riegos del Ambroz, y en otros trabajos realizados o en ejecución. El conjunto de las obras fue ejecutado entre 1988 y 1992. Hasta el año 2007, los trabajos de explotación y mantenimiento de estas presas, han venido siendo desarrollados exclusivamente por el personal de la Confederación Hidrográfica adscritos al área de Proyectos y Obras, si bien, como en otros muchos sitios, desde entonces se cuenta con una empresa de servicios para realizar labores de apoyo a la explotación, mantenimiento y conservación.

1 EL SISTEMA AMBROZ Y LA PRESA DE BAÑOS COMO EJEMPLO SIGNIFICATIVO

1.1 *Introducción*

El sistema de riegos del Ambroz se sitúa en el norte de la provincia de Cáceres, siendo el conjunto de las obras ejecutado entre 1988 y 1992.

Hasta el año 2007, fecha en que se inició el contrato de servicios de Explotación, Mantenimiento y Conservación de las presas, estos trabajos habían venido siendo desarrollados exclusivamente por el personal de la Confederación Hidrográfica del Tajo adscritos al área de Proyectos y Obras.

La antigüedad de las obras que componen el sistema no es muy alta en comparación con la edad media del parque de presas españolas, si bien, debido a que desde su construcción hasta el año 2007, el gasto habido en mantenimiento y conservación ha sido muy reducido, las presas han ido perdiendo funcionalidad y seguridad en sus instalaciones y órganos de desagüe.

En octubre de 2007, la Dirección General del Agua del Ministerio de Medio Ambiente adjudicó el referido contrato de servicios de Explotación, Mantenimiento y Conservación de las presas de Baños de Montemayor, Azud de Hervás, Aldeanueva y Villar de Plasencia, con un presupuesto de adjudicación de 776.046,12€ (IVA exc.) y por un periodo de dos años (prorrogable de mutuo acuerdo por un periodo igual) a la UTE formada por las empresas Técnica y Proyectos S.A. (TYPSA) y GRUPISA Infraestructuras S.A.

La experiencia en los casi tres años de contrato unido a otras experiencias como la actual puesta en carga y mantenimiento de la presa de La Colada en la provincia de Córdoba, nos permite realizar algunas reflexiones de carácter general sobre este tipo de trabajos que deben ser objeto de reflexión de cara a la optimización de los mismos por parte de las Administraciones.

Por otro lado, respecto a este contrato, se incluyen algunos detalles de los trabajos realizados que, debido a las características de estas presas en las que las necesidades de explotación son relativamente pequeñas unido a que en la presa de Baños hay dos trabajadores de la Confederación asignados a la misma, se han centrado en la implantación de un Plan de Mantenimiento Programado y en la actualización de las instalaciones, algunas anticuadas y otras, muy deterioradas por las condiciones ambientales.

1.2 *Descripción de las presas*

El sistema de riegos del Ambroz consta de un azud en Hervás, sobre el río Ambroz, de derivación de caudales hacia el embalse de Baños de Montemayor que realiza la regulación, y la presa de Aldeanueva o azud de Abadía, también sobre el río Ambroz, para derivación de los caudales regulados por el canal de riego.

El elemento principal del sistema es el embalse de Baños, de más de 40 hm^3, situada en una pequeña cuenca lateral del río Ambroz, sobre el río Baños, y que se alimenta principalmente de

los caudales derivados desde el azud de Hervás, a través de un canal de algo más 3 kilómetros de longitud con capacidad de 4 m³/s. Los caudales soltados por la presa de Baños, se incorporan de nuevo al río Ambroz para ser derivados hacia la zona de riegos en el Azud de Abadía por medio de un canal primero y tubería después, para el riego de 3200 ha.

El azud de Hervás se ubica en el río Ambroz, al oeste de y muy cerca del casco urbano de la población de Hervás. Se trata de una presa de gravedad tipo vertedero, ya que el aliviadero ocupa la mayor parte de sus 81,20 m de longitud de coronación. Su altura máxima sobre cimientos es de 11,60 m. Cuenta con dos tomas en la margen derecha, formadas por conductos rectangulares de 1,50 × 0,90 m² y reguladas por compuertas de tablero de accionamiento electro-mecánico, que vierten las aguas a un canal de trasvase, hacia la presa de Baños de Montemayor. Dispone de un desagüe de fondo compuesto por dos compuertas análogas a las de las tomas y situadas también en el estribo derecho entre el aliviadero y la toma. La clasificación de la presa en función del riesgo potencial es C.

La presa de Baños está emplazada en el río del mismo nombre, afluente por la margen derecha del río Ambroz en el que desemboca. Tanto la presa como el embalse creado se ubican en el término municipal de Baños de Montemayor (Cáceres). Se trata de una presa de escollera con núcleo vertical de arcilla de 52,30 metros de altura sobre cimientos y de 1.228 m de longitud de coronación. El embalse creado por la presa tiene una capacidad de 40,86 hm³ a Nivel Máximo Normal (NMN). El embalse del Baños está dotado de un aliviadero de superficie, fuera del cuerpo de presa, ubicado en la margen izquierda. Consta de un único vano de 20 m de longitud de labio fijo y un canal de descarga que vierte los caudales desaguados a un arroyo afluente al río Ambroz. La presa dispone de un desagüe de fondo que hace también las veces de toma para riegos, compuesto de dos conductos metálicos de 1.200 mm y otro de 700 mm de diámetro. La clasificación de la presa en función del riesgo potencial es A.

Figura 1. Presa de Baños.

La presa de Aldeanueva se ubica también en el río Ambroz, afluente por la margen izquierda del río Alagón. Tanto la presa como el embalse están dentro del término municipal de Aldeanueva del Camino (Cáceres). Se trata de una presa de gravedad de 15,40 metros de altura sobre cimientos y de 157 m de longitud de coronación. El embalse creado por la presa tiene una capacidad de unos 116.000 m³ a nivel máximo normal. El aliviadero es central sobre el cuerpo de presa, que consta de cuatro vanos de 11 metros de longitud. El desagüe de fondo está situado en el estribo izquierdo junto al aliviadero, y consta de dos conductos metálicos de chapa de 1,25 × 1,50 m² dotados cada uno de una compuerta deslizante en el paramento de aguas arriba. La toma para riegos se sitúa también en su estribo izquierdo, a continuación del desagüe de fondo, y está formada por dos conductos rectangulares de chapa de 1,87 × 1,66 m² regulados por sendas compuertas deslizantes similares a las del desagüe de fondo. La clasificación de la presa en función del riesgo potencial es A.

La presa de Villar de Plasencia se sitúa en la ladera oriental de la sierra de Cabezabellosa, y embalsa las aguas del Barranco de la Oliva, afluente del río Jerte por su margen derecha. Se sitúa dentro del Término Municipal de Villar de Plasencia, del que toma su nombre, estando situado el pueblo homónimo a un kilómetro aguas abajo de la presa. La presa es de gravedad, de planta recta, con taludes aguas arriba 0,05:1 y aguas abajo 0,75:1. La coronación tiene 104,50 metros de longitud y 1,40 m de ancho, estando a cota 612,55. La altura sobre cauce es de 16,85 m., siendo la altura sobre cimientos de 19,85 m. El aliviadero es de labio fijo y perfil Creager, con longitud de 21,90 m, distribuida en cinco vanos. El desagüe de fondo está constituido por dos tuberías de fundición y de diámetro 500 mm. La clasificación de la presa en función del riesgo potencial es A.

2 TRABAJOS REALIZADOS

2.1 *Explotación*

Como se ha indicado, los trabajos específicos de explotación de en estas presas no suponen una carga de trabajo relevante, dado que no tienen compuertas en el aliviadero ni la cuenca aportante produce avenidas que exijan un control de niveles exigente durante la época de lluvias. A este respecto, la única presa que dispone de niveles máximos estacionales es la presa de Baños, si bien,

Figura 2. Presa de Aldeanueva.

al estar en derivación, no presenta dificultades para su cumplimiento.

Además, en la presa de Baños hay dos trabajadores de la confederación asignados a la misma que se encargan de la suelta de caudales a río Ambroz para el riego, la maniobra de la toma del canal de alimentación en el Azud de Hervás y la toma de datos de explotación (niveles, caudales, etc.).

Por tanto el trabajo del personal de la UTE se centra en estas presas en apoyar al personal de la confederación cuando es necesario, y en realizar las maniobras de las tomas y los desagües de fondo en las presas de Aldeanueva y Villar de Plasencia, registrando además los datos de explotación de las mismas.

Dentro de los trabajos de explotación, se incluye la auscultación de las presas, basada en los equipos recuperados en la presa de Baños, o el equipamiento mínimo instalado en las presas de Aldeanueva y Villar. La auscultación se centra en la colimación topográfica, medida de asientos ya sea topográfica o mediante células, la medida de subpresiones en el cimiento y el aforo de filtraciones.

Toda la información recopilada, tanto por el personal de la UTE, como por el personal de la Confederación, completado con los datos de aforo y meteorológicos facilitados por el SAIH, permiten la elaboración de las Memorias Anuales de Explotación y los Informes Anules de Auscultación.

Para la realización de las labores de explotación no se dispone de un personal dedicado exclusivamente a las mismas, realizando el mismo otros cometidos de forma simultánea, de acuerdo a su cualificación profesional y a las necesidades de las presas.

2.2 *Mantenimiento programado*

Se ha optado por el término "mantenimiento programado", dado que en este tipo de obras en las que no hay una producción asociada al funcionamiento de máquinas de manera continua, el mantenimiento predictivo, preventivo y pequeño correctivo se confunden. La medición del desgaste no se realiza por horas de funcionamiento o unidades producidas, sino atiende a criterios de seguridad y garantía en su funcionamiento, independientemente del número de maniobras realizadas o del desgaste óptimo desde el punto de vista económico.

Además, la gestión del mantenimiento y conservación se realiza de forma conjunta, dado que ambas actividades están íntimamente relacionadas, tanto desde el punto de vista de los elementos objeto de ambas actividades, como desde el punto de vista de la posible relación entre uno y otro. Las operaciones de conservación, no dejan de ser mantenimiento correctivo ya que pueden estar provocadas por el resultado de las operaciones de mantenimiento programado.

Para la planificación y gestión del mantenimiento, se utiliza una base de datos que permite gestionar el inventario de equipos e instalaciones y su mantenimiento, tanto programado, generalmente de tipo preventivo, y el correctivo; ya sea originado por una avería o por una deficiencia detectada durante las inspecciones u operaciones de mantenimiento, o se trate de una obra de mejora de las instalaciones en lo que respecta a su funcionalidad, su seguridad en la operación y/o frente a terceros, o reducir los costes de mantenimiento.

El mantenimiento programado parte de un inventario completo de todas las instalaciones y equipos, que se materializa en una descripción, una ficha de características, fotografías y documentación técnica, estructurado todo ello en una base de datos en Microsoft Access.

Cada equipo o instalación, están agrupados funcionalmente mediante varios niveles, en forma de estructura arbórea, lo que permite un óptimo acceso a la información. Los niveles a utilizar corresponden a: la presa (Nivel 1), el grupo (órganos de toma/desagüe, obra civil, instalación eléctrica, etc.), el conjunto (Nivel 2), dentro de los órganos de toma/desagüe se pueden considerar: toma de riegos, desagüe de fondo, etc., un subconjunto (Nivel 3), para el caso que la estructura de la información lo requiera, p.e., cuando las válvulas de un órgano de desagüe se encuentran en dos cámara diferenciadas), y un ultimo nivel que representa el elemento o equipo (Nivel 4), que es objeto del inventario y sobre el cual recaen las operaciones de mantenimiento, lo que no quiere decir, que la una operación de mantenimiento programado no recaiga simultáneamente sobre varios equipos o elementos similares. Todos los elementos del inventario cuentan con una fecha de alta y baja para permitir la actualización del mismo según se realizar obras de mejora sin perder la información histórica de los equipos sustituidos.

Para cada tipo de equipo o elemento se definen una serie de operaciones de mantenimiento programado, pertenecientes a las diferentes gamas establecidas (eléctricas, mecánicas, otras). Cada operación esta compuesta de una descripción, detalle de las comprobaciones a realizar, frecuencia, mes de inicio, fecha de alta, herramientas y repuestos necesarios y medidas de prevención de riesgos. El mes de inicio, tiene por objeto adecuar el mes de realización a las condiciones de la explotación o climatológicas necesarias para su realización.

Cada elemento o equipo sujeto a mantenimiento se vincula con la operación de mantenimiento mediante una relación viva, es decir, con fecha de alta y baja, posibilitando la baja temporal o suspensión temporal cuando la realización de reparaciones del elemento en cuestión, o la realización de obras o mejoras en las inmediaciones impida realizar la operación.

La planificación del mantenimiento programado se realiza por periodos mensuales para facilitar las labores de gestión correspondientes, lo que no impide que una operación de mantenimiento recoja las diferentes pruebas semanales de un equipo, como en el caso de los grupos electrógenos o los sistemas de comunicación.

Con la información del inventario y la planificación del mantenimiento programado, se genera un cronograma de actividades mensuales en las que se utiliza una combinación de colores y letras para identificar el estado de cada operación de mantenimiento:

Es evidente que en estos contratos no es posible atender los trabajos especializados con personal propio, ya que para poder disponer de un electricista o un mecánico es necesario un grupo importante de presas que den una carga de trabajo suficiente, y aún así, es difícil atender todos los trabajos de mantenimiento programado debido a la importante especialización de la mayoría.

Tabla 1. Claves para las operaciones programadas de mantenimiento.

Clave		Significado	
Color	Letras	del Color	del Código
Azul	–	Prevista	–
Amarillo	–	En curso	–
Rojo	–	Realizado	Deficiencia detectada
Verde	–	Realizado	Sin incidencias
Verde	DL	Realizado	Incidencia menor
Verde	AC	Realizado	Deficiencia resuelta mediante Acción Correctora
–	ST	Programada	Suspendida temporalmente hasta que se realice la acción correctora

Debido a ello, las operaciones de mantenimiento programado que superan la mera comprobación de los equipos (mensual), en los campos mecánico, eléctrico, electrónico, etc. se subcontratan a empresas especializadas de la zona, que además se encarga de resolver las averías y mejoras en caso de producirse.

El equipo propio de mantenimiento está destinado a las labores generales sin especial cualificación: limpieza, tratamiento anticorrosivo de carpinterías y elementos metálicos, conservación de edificios, desbroce, albañilerías, etc. Estos trabajos unidos a las labores de apoyo a la explotación y auscultación, y la vigilancia a pie de tajo de las actuaciones de mantenimiento correctivo que se estén realizando con medios externos, suponen una carga de trabajo adecuada para cuatro-cinco personas en un conjunto de cuatro presas.

2.3 *Mantenimiento correctivo*

De cara a la gestión del mantenimiento correctivo, las actuaciones se clasifican en tres grupos según el origen de las mismas:

- Actuaciones derivadas de incidencias en el Mantenimiento Programado.
- Actuaciones derivadas de deficiencias observadas en las labores de explotación e inspección.
- Actuaciones de mejora de las instalaciones.

Las actuaciones, y por tanto el mantenimiento correctivo, tienen una componente muy fuerte de labores de ingeniería, ya que es necesario:

Figura 3. Presa de Baños. Galería en fondo de saco para conductos de desagüe de fondo y toma.

– Detectar correctamente la patología a corregir, que en casos como el de la cimentación, no son evidentes de resolver.
– Establecimiento de las prioridades de acuerdo a criterios de seguridad y teniendo en cuenta la disponibilidad presupuestaria.
– Preparar las especificaciones técnicas, planos, presupuestos, etc. para su ejecución.
– El control de su ejecución de acuerdo a las especificaciones y diseños.
– Documentación de las actuaciones mediante la preparación de documentos "as built" y la actualización del inventario, del archivo técnico de la presa y de los documentos de seguridad, en caso de que sea necesario.

Estas necesidades, unidas a la carencia de personal asignado a la explotación y de tiempo disponible para atender las presas, lo cual es extensible a la mayoría de los organismos y titulares públicos y privados, se ha reforzado estos aspectos en el equipo asignado al Contrato.

Por otro lado, dadas las limitaciones presupuestarias de estos contratos, en la mayoría de los casos el trabajo se limita a la preparación de especificaciones y valoraciones para la preparación de expedientes de obra menor y negociados, y a la realización de una supervisión de las obras ejecutadas para el cumplimiento total de los objetivos perseguidos por la actuación.

Por otro lado, se ha conseguido poner en marcha varias actuaciones por medio del Servicio de Aplicaciones Industriales, que ha realizado los contratos necesarios para llevar a cabo las mismas.

En las diferentes presas se están realizando numerosas actuaciones, siendo las más destacables:

– Recuperación y mejora de la auscultación de la presa de Baños: nivelación, Piezómetros de Cuerda Vibrante (PCVs), células hidráulicas de asiento, aforadores, etc.).
– Reparación y protección de algunos tramos del camino perimetral del embalse de Baños.
– Instalación de escala de nivel, piezómetros hidráulicos, aforadores y sistemas de colimación y nivelación en las presas de Aldeanueva y Villar de Plasencia.
– Revisión integral y motorización de las compuertas de toma y desagüe del azud de Hervás.
– Sustitución/motorización de las válvulas compuerta de desagüe de fondo de la presa de Villar de Plasencia.
– Reparación de las compuertas de la toma del canal de riego en la presa de Aldeanueva y sustitución del cuadro de maniobra.
– Suministro alternativo a la presa de Villar de Plasencia mediante un grupo electrógeno.
– Mejora del acceso, limpieza del entorno y cerramiento de la presa de Villar de Plasencia.

Figura 4. Presa de Villar de Plasencia. Estado del desagüe de fondo antes de la actuación.

Figura 5. Presa de Villar de Plasencia. Desagüe de fondo después de la actuación.

– Mejora de la toma de agua de abastecimiento y riego mediante un juego de válvulas que permita dar servicio de forma independiente desde ambos conductos del desagüe de fondo.

Adicionalmente se han estudiado, valorado y definido las siguientes actuaciones que están pendientes de dotación presupuestaria:

– Mejora del acceso y camino perimetral de la presa de Baños.
– Revisión integral de los órganos de toma y desagüe de la presa de Baños.
– Instalación de aforadores en las diferentes tomas de la presa de Baños para el control de las demandas.
– Ventilación forzada de la galería de Baños, para reducir el nivel de humedad para mejorar la conservación de los equipos electromecánicos y mejorar las condiciones de seguridad eléctrica.
– Mejora del pavimento y de la seguridad vial en el camino de servicio del canal de alimentación de Baños.

– Instalación de un segundo cierre en los conductos del desagüe de fondo de la presa de Aldeanuela.
– Dragado de unos 20.000 m³ de sedimentos del vaso del embalse de Aldeanueva.

3 ASPECTO RELEVANTES

3.1 *Presupuesto*

De acuerdo a lo indicado en los antecedentes, dado que el contrato corresponde a cuatro presas y tiene una duración de dos años, el presupuesto (adjudicado) por presa y año asciende a aproximadamente 97.000€ una vez descontado el IVA.

Es evidente, que este ratio es más o menos pequeño en función del número de presas considerado dentro de un mismo paquete, de las cantidades de equipos o elementos a mantener y del estado inicial de los mismos.

En el caso de contratos en los que se agrupan 8 o 10 presas en un radio razonable, ya sea en distancia o en tiempo, hay una importante economía de medios materiales y humanos. Además, las partidas de actuaciones correctivas, se pueden gestionar mejor al ser más voluminosas en su conjunto y poderse establecer prioridades en el conjunto de presas.

En cualquier caso, no hay que olvidar, el estado de conservación de las instalaciones y sus necesidades de conservación, ya que en muchos casos, suponen un coste muy superior a las cantidades asignadas a estos contratos, y por tanto el presupuesto disponible no permite ni siquiera mantener unos niveles mínimos en la funcionalidad de los equipos y elementos, entendiéndose la funcionalidad como el compendio de operatividad, garantía de funcionamiento, seguridad para el personal y frente a terceros, etc.

De la experiencia acumulada, se considera necesario un mínimo de 25.000€/mes (IVA exc.) para disponer de un equipo adecuado (ver apartado 3.2) y medios suficientes para atender un conjunto de cuatro presas en un radio de 20–30 kilómetros, en lo que respecta al conjunto de actividades, variando poco al añadir o reducir alguna presa.

En lo que respecta a las partidas para mantenimiento programado especializado y correctivo, salvando la limitaciones indicadas, se considera conveniente un mínimo de 40.000–60.000€ por presa y año, dependiendo de sus características, al menos en los primeros 5 o 10 años de contrato para la puesta al día de las instalaciones.

De estas cifras, se deduce que el presupuesto anual por presa, para un contrato con cuatro presas similares a las que son objeto principal de análisis, debería alcanzar al menos la cifra de 125.000€. Como se ha indicado, el presupuesto podría reducirse pasados unos años vez puestas al día las instalaciones.

En cualquier caso, para ajustar las necesidades de una presa, hay que tener en cuenta otros aspectos no mencionados y que supondrían en todos los casos aumento del presupuesto, como pueden ser presas en las que hay que mantener el Sistema de Aviso a la Población para el Plan de Emergencia, si incluye largos caminos o numerosos edificios de explotación.

3.2 *Personal*

Teniendo en cuenta las labores habitualmente necesarias en las presas, y de acuerdo a la experiencia obtenida, para un conjunto de hasta 4 presas, se considera necesario un equipo de mantenimiento general, con los cometidos indicados en el apartado 2.2, formado por 2–3 trabajadores de baja cualificación que apoyen en las labores de explotación y auscultación. Además es necesario un Ingeniero con experiencia en presas que dirija el equipo, panifique el mantenimiento programado y defina las actuaciones necesarias junto con el responsable de oficina técnica, el cual deberá tener conocimientos de auscultación de presas para llevar a cabo dichos trabajos.

Cuando el número de presas es inferior a tres, la dotación no se puede reducir, pudiéndose reducir la dedicación, si bien esta solución dificulta la prestación del servicio por la irregularidad temporal del mismo y no abarata sustancialmente el coste.

Teniendo en cuenta los ratios económicos por presa manejados por el Ministerio para este tipo de contratos, parece que el óptimo sería agrupar entre 6 y 10 presas, dependiendo de la entidad de las mismas y siempre que estén en un ámbito geográfico razonable para cubrirlo desde un centro de conservación centralizado. De esta forma, se puede disponer de un equipo de mantenimiento general por cada 2–3 presas y los equipos especializados más necesarios rotando entre las presas o en itinerancia.

Esta organización, que trata de optimizar los recursos económicos, no exime al Titular de disponer de un Encargado de presa y del personal de apoyo que sea necesario en función de las necesidades de explotación, como es el caso de presas de embalse con una exigencia alta en la laminación de avenidas, en las que el personal de la contrata únicamente podrá prestar apoyos puntuales en esta labor, debido a que su dedicación esta repartida entre varias presas y principalmente dirigida a las labores de mantenimiento.

3.3 *Capacidad*

De nuestra experiencia, podemos concluir que los trabajos objeto de la Explotación, Mantenimiento

y Conservación de una presa es una labor muy especializada y compleja que requiere una capacidad técnica y experiencia elevada del personal, siendo mucho menos importante la capacidad de movilizar recursos y organizarlos para alcanzar la máxima productividad al menor coste.

En base a esta reflexión entendemos que este tipo de contratos de servicios, tan como se plantean en su gran mayoría, están mucho más próximos a un trabajo de consultoría de ingeniería que de construcción, lo cual es entendido así en algunas Confederaciones y otros organismos.

La principal vinculación de estos contratos con la actividad de construcción, es la necesidad de subcontratar las actuaciones a realizar en las presas, dado que en muchos casos corresponden, o están asimiladas, a actividades de construcción, y por tanto, de acuerdo a la legislación vigente, no es posible hacerlo por empresas de ingeniería.

Desde el punto de vista económico, se optimizaría el gasto si las actuaciones fueran contratadas directamente por el Titular con el apoyo técnico y administrativo del consultor, ya que se reduciría el coste de la intermediación del contratista, realizando la labor de supervisión de las obras como asistencia al Director del contrato. Es cierto que esto requiere establecer las correspondientes partidas por parte del Cliente para su gasto mediante contratos menores o negociados, verificando el homogéneo uso de los mismos, y que en el caso de las Administraciones Públicas puede retrasar la ejecución de los trabajos por cuestiones burocráticas.

3.4 *Prevención de riesgos laborales*

Desde el punto de vista normativo, las actividades de mantenimiento y conservación se encuentran asimiladas a las de construcción por el Real Decreto 1627/1998 de Prevención de Riesgos Laborales en las obras de construcción, y por tanto resulta de aplicación. Actualmente, los contratos de conservación de presas del Ministerio, lo tienen en cuenta, ya que incluyen las labores de coordinación de seguridad y salud.

Este planteamiento adolece de un defecto que reduce la efectividad de dicha normativa, y es que el coordinador pertenece a la contrata, por lo que esta se controla a si misma el cumplimiento del Plan de Seguridad y Salud, no a la Dirección del contrato como prevé dicho Real Decreto.

A este respecto se proponen dos medidas alternativas que permiten mejorar este aspecto sin producir incrementos de costes:

– En caso de considerar los trabajos de conservación en labores de consultoría especializada y mantenimiento general claramente no especializado, el Consultor podría disponer de un Técnico de Prevención que velase por el cumplimiento del correspondiente Plan por parte de su personal y de las empresas colaboradoras. En caso de que el cliente contratase una actuación de obras o asimilable, el consultor prepararía el correspondiente Plan de Seguridad y Salud además de las especificaciones técnicas necesarias, y dispondría un Coordinador durante su ejecución, dentro de las labores de supervisión a realizar.
– En caso de mantener la estructura presupuestaría actual de los contratos, que puede exigir la realización o subcontratación de obras, las labores de coordinación deberían independizarse en un contrato independiente y que abarcase un mayor número de presas, de forma que adquiriera el volumen de trabajo suficiente para una adecuada prestación del servicio.

  Estos contratos podrían tener ámbito provincial, tal como vienen realizando las administraciones en relación a los contratos de conservación de carreteras, y la asignación presupuestaria se retraería de la partida de coordinación existente en los diferentes contratos que agrupase.

Los aspectos de prevención de riesgos se ven dificultados en los casos en que el adjudicatario es una UTE entre una empresa constructora y una consultora, pudiendo en algunos casos no ser posible el cumplimiento de la Ley de Subcontratación. Esta situación puede darse, por tratarse desde el punto de vista administrativo de un contrato de servicios en los que habitualmente rige la clasificación O-3 que puede ser obtenida tanto por empresas constructoras como consultoras.

4 CONCLUSIONES

De acuerdo a la experiencia obtenida, se han llegado a una serie de conclusiones, las cuales serían objeto de un necesario debate, con el fin de poderlas dar un carácter general, dado las experiencias particulares sobre las que se han extraído:

1. La asignación presupuestaria por presa y año en los contratos del Ministerio es insuficiente para conseguir la optimización de los resultados a alcanzar. Es cierto, que en contratos con un número de presas elevado y según estas se van poniendo al día, las necesidades se acercan más a los ratios actuales. En cualquier caso, estos presupuestos no permiten eliminar del todo el personal de explotación de las presas que tengan unos requerimientos de explotación elevados como es el caso de grandes embalses de laminación de avenidas.
2. El equipo mínimo recomendable para las labores de explotación, mantenimiento y conservación

de presas, consta de un equipo de mantenimiento general formado por dos trabajadores, un Ingeniero especialista en explotación de presas y un técnico para labores de oficina técnica y auscultación.

3. La seguridad de las presas requiere una capacitación técnica elevada propia de empresas consultoras de ingeniería, más que la capacidad de movilización y organización de recursos propia de las empresas constructoras. De acuerdo a este esquema, las actuaciones de obras en las presas serían contratadas directamente por el Titular y supervisadas por la empresa consultora adjudicataria de los trabajos.
4. Desde el punto de vista de la prevención de riesgos, es necesario replantear el esquema vigente para dar total cumplimiento y alcanzar los objetivos de la normativa vigente.
5. Estos contratos han permitido en muchos casos poner en práctica la mayoría de los aspectos de las Normas de Explotación, detectándose numerosas carencias o imprecisiones que es necesario corregir, además de adaptarlas a esta nueva situación que basa una parte importante de las tareas en recursos ajenos al propio titular. A este respeto, hay que señalar que en la mayoría de los casos, las dotaciones de personal conjunto no alcanza los requerimientos de dichas Normas.
6. Se considera que los grupos de trabajo dentro de los organismos públicos gestores de presas, compuestos por los ingenieros encargados y las empresas especializadas, es la mejor herramienta para la optimización de los recursos económicos en conservación, el seguimiento de unos criterios unificados, y la obtención de unos resultados más homogéneos en todos los contratos simultáneos o sucesivos.

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Management of the maintenance and conservation of the dams of the Ebro Hydrographic Confederation

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ABSTRACT: Currently, the Ebro Hydrographic Confederation is managing the property of 53 dams (including those in the construction phase) with an average age of 50 years. Described in detail here are the structure and resources this autonomous organism has to carry out the maintenance and conservation of their structures in compliance with current regulations. Also described is the recent implementation of the computer portal called GEISER (Management of Reservoirs, Inspection, Safety, Equipment and Inspections) which will manage the safety of state-owned dams and reservoirs, structured in auscultation, the maintenance of the drainage system, the management of the reservoir levels (also floods) according to the Operating Standards, the activation of the Emergency Plan, the technical file and the technical specifications.

1 INTRODUCTION

The Ebro Basin holds a hydraulic treasure that is the heritage of our ancestors dating back to the first century of our era in the times of the Roman Empire with examples such as Almonacid de la Cuba, Muel, Puy Foradado, Azud de la Rechuela and Presa de la Pesquera in Zaragoza, the Virgen del Pilar Hermitage, Parede de los Moros and Villafranca in Teruel, and Andelos and Odrón in Navarre. After a long period would come the medieval dams of *calicanto* or masonry work consolidated with lime mortar from the 15th and 16th centuries, from the basin of the Aguas Vivas (Malpasillo, possibly the first "thin-arch" dam in Spain, El Vado, Galindo, Azud de Moneva, La Estanca, etc.). Arguís (Huesca, 1687–1704) subsequently heightened by Manuel Lorenzo Pardo in 1929, the old Mezalocha (Zaragoza, 1743) ruined due to piping and subsequently rebuilt.

The advances at the end of the 19th century made possible the construction of dams on very full-flowing rivers such as the Ebro; this is the case with the Azud de Pignatelli (Navarre, 1790) and Escuriza (Teruel, 1880–1896).

At the beginning of the 20th century, masonry work dams were built with cement mortar from Santa María Belsué (Huesca, 1903–1918) Moneva (Zaragoza, 1909–1929), Cueva Foradada (Teruel, 1912–1929) La Peña (Huesca, 1913).

With the Camarasa dam (Lleida, 1919) began the first concrete dams (cyclopean concrete) in Spain, which would be followed by Pena (Teruel, 1924–1930), Gallipuén (Teruel, 1927), Barasona (Huesca, 1928), Moneva (Zaragoza, 1929), Cienfuens (Huesca, 1931), Santolea (Teruel, 1932) Ardisa (Zaragoza, 1932) built with these same concretes. In 1930 the Alloz dam (Navarre) was the second great vaulted dam built in Spain.

There was a sudden stop after the Civil War; only the Ebro Dam (Cantabria, 1945) and Las Torcas Dam (Zaragoza 1948) were built. Recovery came with the two decades of most dam building in Spain during the 1960s and 1970s, mainly utilizing conventional concretes: Mediano (Huesca, 1959, heightened in 1973), Yesa (Navarre, 1959), Oliana (Lleida, 1959), La Tranquera (Zaragoza, 1959), Mansilla (La Rioja, 1960), Santa Ana (Huesca, 1961), González Lacasa (La Rioja, 1962), Mequinenza (Zaragoza, 1964), El Grado (Huesca, 1969), Búbal (Huesca, 1971), Vadiello (Huesca, 1971), Eugui (Navarre, 1971), Lanuza (Huesca, 1978), Guara (Huesca, 1995).

The first dam constructed out of loose materials was Almochuel (Zaragoza, 1914) followed by

Estanca de Alcañiz (Teruel, 1944), La Sotonera (Huesca, 1963), Caspe (Zaragoza, 1963), Maidevera (Zaragoza, 1981), Calanda (Teruel, 1982), Monteagudo de las Vicarías (Soria, 1982) Valdabra and Torrollón (Huesca, 1983), Mairaga (Navarre, 1990), Pajares (La Rioja, 1994),

Then came the roller-compacted concrete (cold concretes) dams: Urdalur (Navarre, 1994), Val (Zaragoza, 2001) Rialb (Lleida, 2000) and currently Enciso (La Rioja, in construction).

Recently, dams have been built where their foundations posed geotechnical challenges: Montearagaón (Huesca, 2004) Itoiz (Navarre, 2005), Lechago (Teruel,) Valcomuna and La Trapa (Teruel, 2005) La Loteta (Zaragoza, 2009).

The Ebro Hydrographic Confederation is responsible for the management of all the state-owned hydraulic property, currently 53 reservoirs. Included here are the four currently under construction (Mularroya, Enciso, Soto-Terroba, and Las Fitas), because the safety of the dams begins at the start of their construction.

The types are 40% gravity (21), 34% loose materials (19), 8% roller-compacted concrete (4), 6% vaulted (3), 6% arch-gravity (3), 2% rock fill with impervious core (1), 2% asphalt faced (1) and 2% mixed (1).

The average age of dams currently in operation is 47 years.

The Confederation is structured in 17 Operating Boards which are managed with the following human resources: the Deputy Manager-Head of Operation, 6 Department Heads, 8 Section Heads, 28 Supervisors and 95 Technical Activities Technicians. Also contributing in the conservation and maintenance of the dams: the Auscultation and Geotechnics Service, the Automatic System of Hydrological Information Service (SAIH), the Industrial Applications Service and the Topography Section.

For years, the Ebro Hydrographic Confederation (CHE) has had the aim of implementing in its structure computer tools to aid in the management of safety and the maintenance of the dams and reservoirs it operates, in compliance with the Operating Standards, and the Emergency Plan of each reservoir.

The computer portal developed is called GEISER (Management of Reservoirs, Inspection, Safety, Equipment and Inspections) and is housed in a server of the central offices of the CHE in Zaragoza. Through the communications network of the Automatic System of Hydrological Information (SAIH Ebro) from each of the 53 reservoirs the server is accessed, supplying the data in real time. Each user accesses the portal through the intranet with a different user profile depending on their functions; they may also access from any

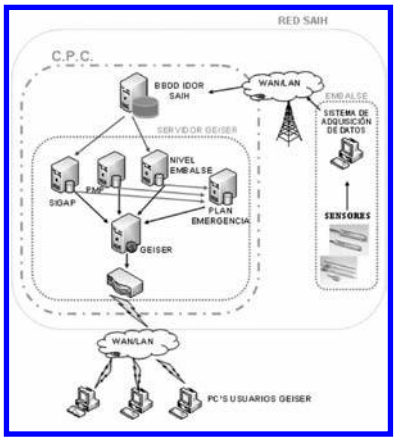

Figure 1. Diagram of communications of the GEISER.

place through VPN connections from internet with the pertinent permission and security control.

An important tool of the portal is the management of the notifications generated by each of the computer applications, informing each user, for example, of the maintenance operations to be carried out in the near future, the monitoring readings, the occasional exceeding of thresholds, flood notifications, entering emergency scenarios, etc.

The computer portal is made up of six applications called SIGAP, PMP, Reservoir Levels Management, Emergency Plan, Technical File and Technical Specifications. Its main characteristics are stated in the following points.

2 INTEGRAL MANAGEMENT SYSTEM OF DAM AUSCULTATION (SIGAP)

In collaboration with the Centre for Studies and Experimentation of Public Works (CEDEX), a computer application has been developed that manages the auscultation of the reservoirs, called SIGAP. The application captures the automatic auscultation data that arrive at the Basin Processing Centre (CPC) of the Automatic System of Hydrological Information (SAIH), together with the manual auscultation data. The SIGAP filters them through a statistical process to eliminate reading errors, stores them and launches notification messages if certain pre-defined thresholds are exceeded. The programme indicates the user the planning of periodical manual readings he must make at all times, as well as the inspection and maintenance of the instrumentation.

At any moment, reading is made possible in real time, displaying the dam information in SCADAS (Supervisory Control and Data Acquisition), of both the current and the historical data.

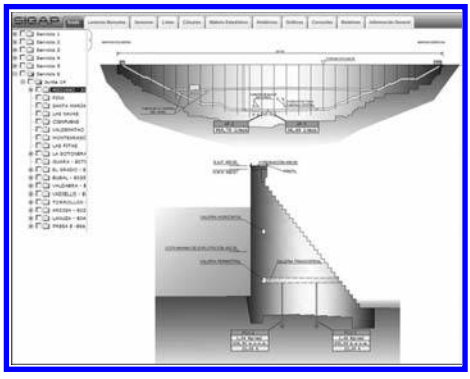

Figure 2. Main screen of the SIGAP programme.

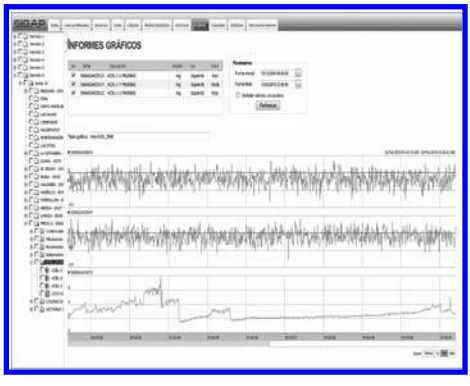

Figure 3. Example of auscultation graphs: accelerometers.

This information can be viewed through time-line graphs, as well as correlating different variables. Likewise, the application centralizes topographic, inclinometric and seismic data.

The application has an information module in which the different behaviour thresholds of each sensor are established, categorizing them as thresholds of operation, safety and emergency scenarios according to the contents of the Emergency Plan of each dam.

In a more advanced way, the application allows the statistic modeling of the data, with which behavioural models can be predicted or clarified and the determination of thresholds can be facilitated.

Lastly, reports can be generated on request, configuring templates, whether of annual behavioural reports, auscultation notes, etc.

3 DAM MAINTENANCE PROGRAMME (PMP)

The Dam Maintenance programme manages, in compliance with the Operating Standards, preventive

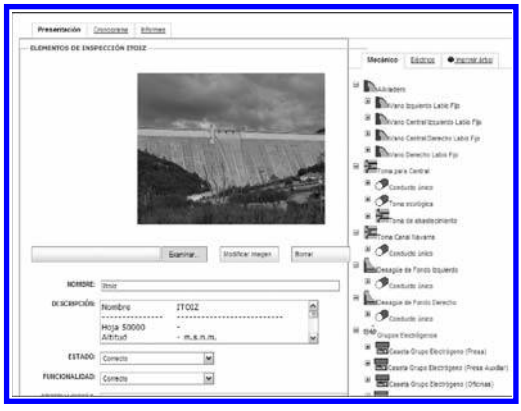

Figure 4. General screen of the PMP application.

and corrective maintenance of the hydro-mechanical equipment of each dam. This application includes all work done after 10 years through the compiling and storage of the bimonthly maintenance booklets.

Three parts are distinguished:

a. Presentation. The software makes it possible, through simple tree-structured navigation, to analyze all the mechanical or electrical elements the dam is composed of. In the splash, screen photographs are displayed and the elements, their status and their functionality are described at a distance.
b. Chronogram. A table has been designed that specifies all the maintenance operation that must be carried out periodically and their execution procedure in compliance with the Operating Standards and current Safety and Health regulations. Thus, the person in charge of each operation can view the tasks completed, pending or programmed. Any authorized user can access these operations at any time and check their status, the observations generated and the element to which this is applicable. This organization allows the Services of Operation and Industrial Applications to have all the information relating to the status of each element in real time, easily and quickly.

   There is a system of automatic communication among the users, so that certain actions (such as postponing of a maintenance operation) generate notifications to those responsible for that action. A smooth-flowing channel is established among the users, with a record of communication to avoid confusion or loss of data.
c. Reports. As there is an important data history, the PMP is able to extract reports on the maintenance of the elements. Thus, it is possible to know the number or percentage of correct, incorrect, and postponed operations, the materials supplied

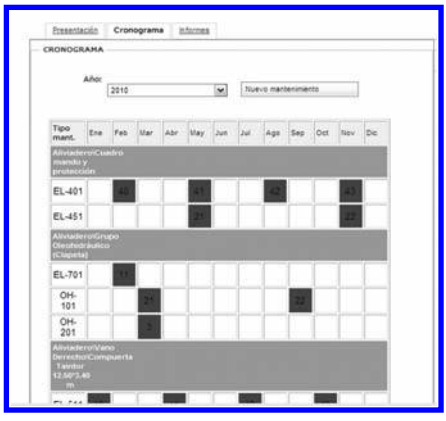

Figure 5. Example of chronogram.

throughout the year, the hours utilized, etc. Finally, the application allows isolated synthesis of the overall status of the drainage system, for inclusion in the mandatory Annual Report.

This application will generate a notification sent to the Emergency Plan module whenever a vital element is out of service.

4 MANAGEMENT OF RESERVOIR LEVELS

The Spanish Technical Regulations on Dam and Reservoir Safety (1996), in article 5.7, indicates that dam owners have to write Operating Standards which must include the safety regulations and which must contain, among other things, the appropriate freeboard in the reservoir during times of flood risk, the operation rules for spillway gates in case of floods and the precautions to be taken so as to prevent the premature discharge of flows that could cause damage downstream, which has led to the inclusion in GEISER of a specific module that deals with all the matters relating the management of reservoir levels, both in ordinary and in flood situations, according to the criteria established in the Operating Rules of each reservoir, although this is not posed as being automatically executable.

The main functionalities of the application are:

a. Record of daily operation: entry, recording and consultation of the daily data of reservoir levels and meteorological parameters by the user.
b. Flood management: Automatic detection of possible flood situations based on real-time readings from the SAIH (reservoir levels, discharge volumes, flow volumes of gauging stations and readings of rain gauge stations); obtaining of the proposal of volumes to be discharged by the spillway during the flood situation, in the absence of higher criteria (decision by the Permanent Committee of Art. 49 of the RDPH)
c. Record of flood hydrographs, with the purpose of generating a history for consultation and application (presented from experience).
d. Automatic communication and notifications to affected users and organisms, once a specific flood situation is activated by the person in charge.
e. Consultation of readings: makes possible consultation of histories of reservoir levels, flow volumes of gauging stations, rainfall in rain gauge stations, drained flows, etc.
f. Annual report, including hydrographs of recorded floods, declarations of flood scenarios, communications issued, statement indicating the maneuvers carried out during the flood, reports of incidents, etc.

The fundamental base on which this application is supported is consultation, in real time, of data from the SAIH of the Ebro, especially indications of reservoir levels and flow volumes in gauging stations and reservoirs. Based on this data, the application is able to detect those factors which determine that the reservoir is in a new situation with regard to floods. The application tells apart various situations: normality, flood notification, flood, damages, simulation mode and test mode.

Once a situation other than normality is detected, those in charge of the dam operation are notified (Operating Service and Permanent Committee), as they have the authority to officially declare a new situation and activate the protocol of communication.

5 EMERGENCY PLAN

The Basic Guidelines of Civil Protection Planning for Flood Risk (1995) and the Technical Safety Regulations for Dams and Reservoirs (1996) establish the obligation of the owners of the dams, classified in categories A or B, to prepare and implement the dam Emergency Plan; for this reason, in recent years, these plans have been roled out in the area of the Ebro Hydrographic Confederation.

As is the case with management of reservoir levels, included in GEISER is a module with the purpose of being a tool to aid in the detection and management of the different scenarios included in the Emergency Plan.

The computer tool is based mainly on the automatic detection of the factors which, according to the Plan, establish that the dam can be in a particular emergency scenario (subject to the formal declaration by the person in charge), the automatic launching of communications, and consultation of the actions necessary to manage

Figure 6. Basic flowchart of operation.

the emergency, both in monitoring and control as appropriate corrective measures in each case.

The computer programme, in order to detect the possible factors of a particular emergency scenario, will consult data in real time from the SAIH, relating to reservoir levels, flow volumes, etc, but also from other modules of the GEISER, such as the SIGAP (surpassing auscultation thresholds) or the Dam Maintenance Plan (example: out-of-service situation of the spillway gates) or of Reservoir Level Management.

The application will show the Plan Director when the circumstances appear that are established as indicated for the declaration of a specific scenario, but bearing in mind that his decision will always be necessary for the activation to take effect. Once the declaration is activated, the necessary communications of the Emergency Plan will be started up.

The main functionalities of the application are as follows (leaving aside the options "simulation" and "test", also available):

a. Detection of scenario: as was mentioned previously, based on the SAIH or on the other applications of the GEISER a scenario is detected.
b. Procedure Rules. Reports on the actions established by the Plan, dealing directly and in a mechanized way with those of communication, notification and alert.
c. Information about the causes and indicators of emergency scenarios described in the emergency Plan.
d. Organization. Describes the human resources, materials, structural organization and functional organization available.
e. Territorial zoning. For information purposes, flood maps due to dam breach are accessed, contained in the Emergency Plan.
f. Emergency Thresholds, for the definition and the consultation of the different thresholds defined in the Plan.

6 TECHNICAL FILE

This application carries out the documentary management of the reservoirs. Users can access all types of documentation from a single location, thus facilitating not only access but also the integrity and unicity of the information. In this way, it is possible to avoid the disadvantages of storage on paper, the proliferation of various digital storage systems and their updating.

The stored documentation will be composed of Annual Reports, XYZT Documents, Operating Rules, Emergency Plans, Safety Reviews, Construction Projects, or various types of documentation relating to the dam, spanning not only the documents that are mandatory to store, but also those that may be of interest for the safety of the dam.

These documents will not be visible to all users, as a system of permits will be established, assigning different levels of access to it. In addition, it will be possible to download in electronic format or to print those documents considered necessary. It will also contain a list of all the documents in the paper file.

Finally, there is a management table for the compliance with the current regulations relating to safety of each reservoir, which provides information about the administrative situation of the documentation.

7 TECHNICAL DATA SHEET

The creation of this computer application responds to the need to present and unify the basic information of the dams existing in the CHE. The information of each dam and reservoir is being normalized to achieve fast, easy and effective access to the existing information, as well as to provide an effective tool for updating of information. The information will be available for all the users of the CHE, but in addition to this, most of the fields will be accessible through internet for the general public.

The software will make possible the storage of descriptive pictures of the reservoir (plants, elevations, aerial views, sections, profiles, details, photographs, hydraulic operation diagrams) and a number of text fields specifying the main characteristics of the same. In this way, the user can have an overall view of the main characteristics of the dam.

In addition, searches can be carried out in the fields entered, in order to filter the reservoirs that meet certain requirements, such as larger dams than a given height.

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Integral communications solutions in dams

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ABSTRACT: Normal dam maintenance and conservation work requires information and communications systems which allow instant, effective processing of the data generated. The geographical situation of dams, far from urban centres, and their own internal organization, with buildings often scattered around, are an impediment to access to communications systems. Confederación Hidrográfica del Guadiana (C.H.G.) has introduced a voice and data access and distribution in Zújar Dam which facilitates conservation, maintenance and operating work.

1 INTRODUCTION

Normal dam operation, maintenance and conservation work can be eased by use of information technologies.

The use of data organization software such as spreadsheets and databases and e-mail distribution of data or the use of SCADA data acquisition software facilitates access to and processing of field data.

The biggest problem is access to data and voice communication distribution services, of great importance in day-to-day conservation work, both for internal communications on the dam and for external communications with other service personnel.

This study examines the possible solutions existing for providing dams with access to telecommunications networks and their distribution in the environment, and tells the experience of the solution introduced in Zújar Dam.

2 ACCESS TO TELECOMMUNICATIONS SERVICES

The common lack of public communications services (landlines, mobile telephony, and DSL) in dams because of their customary rural situation is an impediment to increasing the productivity of the operation and conservation team, or providing access to automatic information systems.

2.1 *Public network access services*

Although GSM mobile voice telephony and GPRS/UMTS mobile data telephony are widely used systems with extensive coverage throughout Spain, because of shadows or weak signals caused by orographic features and/or distance from base stations, they cannot be considered adequate solutions for voice or, especially, data communications, even when the possibility of using local gap-filler repeaters is considered. It should be pointed out that the fact that voice communications can be carried out does not mean that data communications can be established. Local repeaters amplify signals received from adjacent base stations, but it must be taken into account that the greater the bandwidth, the lower the efficiency and sensitivity of the amplifiers, making it more likely that GSM (voice-only) signals are amplified nearly exclusively, while GPRS/UMTS signals are not.

The systems offered commercially by telecommunications operators for use in places where GSM/UMTS coverage is poor are unreliable services like TRAC (Telefonía Rural por Acceso Celular, Rural Telephony with Cellular Access), which provides voice services in the same way as fixed telephone lines, when properly maintained. Experience with regard to this system is negative, because in most cases it is still an analogue system, neglected by telephone companies. Data access over TRAC requires its digital version, not available in all rural areas. Digital TRAC provides a maximum downstream rate of 40 Kbps and 14 Kbps upstream, only 0.667%

of the speed of conventional domestic ADSL. This, together with delays of 2 to 10 s, makes it impossible to use as a data communications system.

Satellite links are another solution offered by Spanish telecommunications operators. These services are far from competitive with landlines, allowing access to public voice and data services with acceptable quality in any part of the world. For voice, they offer simple IDSN-type channels and for data they can produce speeds between 492 Kbps and 4 Mbps. The great drawback of satellite links is their very high price based on fixed quotas and pay-per-use, both for voice calls and data communications, making them a reliable, feasible solution in some cases of emergency situations, but too expensive to cover requirements of normal dam conservation and maintenance situations.

2.2 *Private network access services*

Taking into account that the offices of official organizations are located in urban centres, private distribution solutions using in-house communications networks are feasible.

Urban centres have access to public cable communications networks, which these days are those providing the best performance at the best price, so it is possible to distribute access to both voice and data public networks over own networks to the dams, in rural areas.

This idea gave rise to the study of different communications systems intended for service self-provision (an attractive solution because its introduction may become necessary to comply with current legislation on dam safety and emergency plans). The only way to carry communications from one point to the other is by wireless, using point-to-point radio links.

- *Microwave radio links*: The large number of existing manufacturers offer affordable equipment for up to 50 Mbps of Ethernet throughput, but the great drawback is that the frequency bands in which they work are regulated by the National Frequency Assignment Table, managed by the Secretary of State for Telecommunications and the Information Society (SETSI) of the Ministry of Industry, Tourism and Commerce, to which annual fees for the required frequencies and bandwidths must be paid.
- *WiFi links*: These are economic systems of varying performance depending on the length of the link, the standard to which the equipment is made and other factors. They are licensed to operate with restrictions in transmission power in the free bands between 2.4 and 5 Ghz.

  The most widespread systems at the moment fall under IEEE standard 802.11 g, the theoretical physical speed of which is 54 Mbps with a channel width of 20 Mhz, meaning a spectral efficiency of 2.7 bps/Hz. With bandwidths as large as this, the integrated noise power is greater than narrower band systems, and considering that the most efficient modulation that can be achieved is 64-QAM 3/4 with a signal-noise ratio over 30 dB, the power received by the receiver needs to be much higher than with narrower band systems which, added to the fact that access to the medium is statistical based on busting with conflict mechanisms, means that the maximum Ethernet throughput would be less than half the theoretical physical speed. This, together with attenuation in the distance across the link and the low sensitivity of receivers would provide effective throughputs in links of 10 Km of around 5–6 Mbps.
- *WiMax links*: This kind of system, still not widespread and only recently standardized under IEEE 802.16, with a cost higher than WiFi, but lower than microwave radio links, is coming to be considered an effective, feasible solution for this kind of application. There are versions operating in licensed bands, i.e., regulated by the SETSI, and others working in the open 5.4 GHz band.

  WiMax technology is spectrally more efficent than WiFi. It gives physical speeds of 37.7 Mbps with channelling of 10 MHz, which means a spectral efficiency of 3.77 bps/Hz. Efficiency of access to the WiMax medium is 90%, because of its deterministic, interleaved, conflict-free access method, so that under perfect conditions the Ethernet throughput achieved is 34.4 Mbps. With propagation losses and an effective radiant system, speeds of 20 to 24 Mbps would be achieved over a 10 Km link.

The most complicated part of introducing a self-provided system is the physical infrastructure necessary for implementation of the radio links from the offices of the organization to the target dam (which in many cases could be possible if the infrastructure used to cover the cases established in emergency plans is used).

3 DISTRIBUTION OF TELECOMMUNICATIONS SERVICES

The system explained in the previous section resolves the dam's access to both the organization's own communications networks (corporate Intranet, internal telephone network) and to external services (Internet and public telephony), but not one of the biggest problems to be found in dams, which is the distribution of services within the dam itself.

As has been said, dams are likely to have various, scattered buildings such as workshops, stores, offices and residences, from which it is important to have access to communications services in dam conservation work, and the civil engineering works needed to cable all these buildings together could be very costly.

Wireless communications systems provide an advantage in this respect, as the cost of equipment acquisition and installation is considerably less than that of cabled networks.

Depending on the conditions of the environment, the technologies that can be used for access to communications services are WiFi and WiMax.

In making the decision about the communications systems to be used, environmental parameters such as these need to be taken into account:

- The distance between the points being connected.
- The concentration of vegetation interfering with the links.

Depending on these parameters, the characteristics of the transmission system need to be chosen according to its possibilities:

- WiFi-PtMP (Point to MultiPoint): the signal is radiated 360º in a radius of 300 m with external antennae and in a completely unobstructed environment. This configuration is not very suitable for links between buildings, though it is suitable for data access by mobile devices (PDA, laptops) both indoors and outdoors.
- WiFi-PtP (Point to Point): the signal is radiated in a single direction (radio link). At a frequency of 2.4 GHz, even a thin vegetable layer implies the loss of most of the signal, the working frequency being very close to a whole multiple of the resonance frequency of water. At a frequency of 5 GHz, absorption by vegetation is less of a problem, but free-space attenuation is one, reducing the effective length of the link.
- WiMax-PtMP: like WiFi-PtMP, this radiates 360º, but the theoretical range is much greater. Theoretically, the maximum range of a WiMax-PtMP base station is 7–10 Km, but that would be with 3 sectors of 120º each, i.e., 3 base stations with 3 panels arranged in a circular formation so that each panel (sector) radiates around 120º (360º altogether).

  Coverage of an area with a radius of 10 Km, apart from being excessive for the environment of a dam, is economically not very feasible. The solution to be adopted would be to radiate 360º from a single base station with a single omnidirectional antenna. The maximum coverage radius is so limited to 2–5 Km.

  The distances involved would require line-of-sight (LOS) visibility between the base station and the WiMax subscriber, without which (NLOS) the maximum link length could be reduced to 1–2 Km.

The solution to be adopted will depend on the specific case. Before a decision is made, a study of the visibilities between the points to be linked, the complexity of the network and the orography of the area. As well as surveys, this requires use of different methods for calculation of the propagation to find out the attenuations produced by propagation losses in free space and diffractions caused by obstacles. These calculations are usually supported by GIS-based software to generate the coverage spots or make point-to-point propagation calculations.

These are the essential differences between the different systems:

- Compatibility: WiFi is a very widespread standard and there are many manufacturers supplying equipment at affordable prices which is very simple to install and configure but which have a very limited range even when outdoor directional antennae are installed. WiMax installation and configuration is more complex, but it should provide a higher range and greater bandwidth.
- Bandwidth: WiMax can make links of greater bandwidth than WiFi.
- Stability: WiMax links are more stable than WiFi ones, because of their non-random medium-access method, intelligent re-use of frequencies and OFDM multiplexing with 256 sub-carriers, less vulnerable to reflection and diffraction.
- Cost/coverage compromise: WiMax provides greater coverage than WiFi with more bandwidth, but the fact that WiFi is more economical makes it feasible for short, obstacle-free, point-to-point links.

It can be said that WiFi systems are effective solutions for local network distribution inside buildings, of much greater usability than any other system. There are currently a host of communications devices (telephones, computers, mobile devices) using WiFi standard IEEE 802.11 a/b/g/n as physical access layer.

4 PRACTICAL CASE: TELECOMMUNICATIONS SERVICES ACCESS AND DISTRIBUTION SYSTEMS IN ZÚJAR DAM

At the end of 2008, Confederación Hidrográfica del Guadiana decided on installation of a communications network for access to the organization's

telecommunications services in Zújar Dam, and its distribution throughout various buildings in the complex.

The system was designed and introduced between 2008 and 2009, since when it has been used by the water authority's staff in its dam maintenance and conservation work for voice communications with the rest of the organization and access to data networks.

Until the system was installed, Zújar had unreliable GSM coverage using gap-filler repeaters mounted at various points of the dam to provide voice communications. This service did not allow data communications, because the GPRS signal level was extremely weak and 36 Kbps connections were barely achieved.

4.1 *C.H.G.'s voice and data service access system*

At its office in Don Benito, Badajoz, Confederación Hidrográfica del Guadiana has a voice and data communications centre for its SAIH (Sistema Automático de Información Hidrológica, Automatic Hydrological Information System) and for corporate use. This office has a 4 Mbps ADSL Internet connection and voice communications with the public network and the internal network (extensions in all branch offices: Ciudad Real, Badajoz, Merida and Don Benito).

The system for Zújar Dam's access to C.H.G.'s network was implemented taking advantage of the SAIH trunk communications network which interconnects all the offices through digital microwave radio links.

After geographical analysis to determine the feasibility of introducing the radio link, it was decided that the strategic point for making the link with the SAIH trunk network was the repeater in Puebla de Alcocer (R2-11). The link between R2-11 and Zújar Dam was considered potentially feasible because of the direct line of sight.

Between the Don Benito office and Repeater R2-11 there is a microwave radio link in operation with 16 top PDH level E1 bands (1 E1 band = 2 Mbps).

The radio devices in the Don Benito—R2-11 link were first adapted to take Ethernet traffic, because the Zújar network was designed to be full IP, and 2 E1 bands dedicated to the Zújar Dam's traffic were assigned, i.e., a total of 4 Mbps between the Don Benito office and Repeater R2-11.

The link between R2-11 and Zújar Dam was resolved with implementation of a WiMax point-to-point radio link in the 5 GHz band (unlicensed band).

The total length of the gap is 20 Km, which means free-space attenuation of 135 dB.

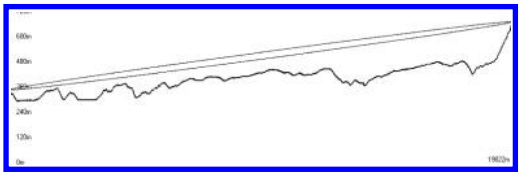

Figure 1. Radioelectric profile between Zújar and R2-11.

Figure 2. Interior WiMax unit in repeater R2-11.

The WiMax radio link was implemented with a WiMax base station in R2-11 and a subscriber unit in Zújar Dam, configured to make the point-to-point link with the greatest possible bandwidth. Both ends are fronted by a directive panel antenna with a gain of 26 dBi.

With the devices' configuration parameters and the gain of the directive antennae, adding extra attenuation due to the average vegetation at the Zújar output towards R2-11, a received signal of between –72 and –76 dBm is achieved. Taking into account that the channelling of these devices is 10 Mhz, making the noise power lower than in systems of greater bandwidth, a signal/noise ratio of nearly 20 dB is achieved, allowing upstream and downstream QPSK ¾ modulations.

The measurements made came to Ethernet throughput speeds of 8 Mbps.

There is a WiMax indoor unit in the Zújar office providing an Ethernet for connection of

Figure 3. WiMax radio link on the television mast of the dam office.

services. A manageable Ethernet switch is added to the port with Layer 3 switching capabilities for service quality management and for connection of computers and IP telephones. The dam office has this been given these connections:

– C.H.G. internal connections: corporate computing system and access to the corporate telephone extensions of the whole organization, not just the Don Benito office.
– External to C.H.G.: connection to Internet and the public telephone system.

The biggest problem, though was distribution to the different sites of interest, as the intention was not only to provide access to telecommunications services from the office, but also from places at a distance from it.

4.2 *Distribution of telecommunications services in Zújar Dam*

The organization's requirements in terms of sites needing access to the dam network established in its office were the starting point.

These are the distances in a straight line between the office and the other sites:

– Office—site 1: 250 m
– Office—site 2: 290 m
– Office—site 3: 63 m
– Office—site 4: 470 m

According to initial estimates with external directional antennae, the best system to introduce would be WiFi point-to-point links between the office and the other 4 sites, but field study showed that the high density of vegetation would be an important obstacle to electromagnetic radiation.

Instead, a WiMax point-to-multipoint system was studied and introduced, with a base station in the office radiating 360º with an omnidirectional antenna with a gain of 8 dBi and a subscriber with an integral 14 dBi directional antenna.

Figure 4. Locations of the site to be connected. The rectangle represents the position of the office and the circles show the points to be connected with it.

Figure 5. WiMax base station at the dam office.

Figure 6. WiMax subscriber unit.

In spite of the vegetation, WiMax is a very effective system, making use of reflections from the ground and building walls, and achieving the maximum bandwidth at each subscriber unit.

The purpose of the subscriber is to translate and transfer the information between the WiMax base station and the interior of the site.

The subscriber unit also has an interior unit mounted inside the site which provides an Ethernet port for connection of services and an adjustable switch like the one in the office to which the computers and IP telephones are connected.

4.3 *Connection and access to services on sites*

As all communications are carried out using IP protocols, voice and data connections have the same interface. At each site, including both remote sites and the office, there is:

- An Ethernet switch for cable connection of computers or IP telephones.
- An intelligent WiFi access point to provide WiFi coverage inside the building, avoiding cabling and so simplifying the connection of devices distributed in different rooms.
- A cable-connected IP telephone for voice communications.
- Fax (office only).

The system is designed so that voice traffic has priority over data traffic (Internet, e-mail, server connection, etc.), so that apart from spontaneous disconnection of links, voice services, being digital, are always high quality.

Figure 7. Ethernet switch at remote site.

Figure 8. IP telephone.

5 CONCLUSIONS

This study examined different wireless solutions of relatively low cost for providing access to voice and data services in places not giving quality coverage by public telecommunications networks, as well as their distribution, i.e., the creation of a local network in the area of the dam.

The system introduced by C.H.G. at Zújar Dam, where the communications were previously unreliable, has been described as being especially noteworthy.

This system has been working for more than a year, and normal dam operation, maintenance and conservation work has been considerably facilitated.

By access to C.H.G.'s corporate voice and data services and external access to public networks, all the sites indicated have a voice communications point (IP telephone), a switch to which a computer can be connected, and WiFi coverage for network access by portable devices.

The services provided by the system have been satisfactory, and C.H.G. may extend it to other dams.

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Geotechnical impact on small earth dams caused by vegetation growth as a result of inadequate maintenance activity

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ABSTRACT: Small dams for irrigation use are often managed by Irrigation Communities in Spain. These groups of landowners, according to the law, are responsible for devoloping routine maintenance actions of each dam and their consequent common irrigation system. Due to various social factors, like population ageing or loss of farming, in areas where there has been a notable decline of agriculture the maintenance activity of irrigation hydraulic net is defectively conducted.

An inadequate practice of maintenance allows the immediate growth of vegetation which may develop over the earth dam embankment, affecting to it in different ways depending on whether herbs, shrubs or even tree roots penetrates in the compacted soil. The problem with trees and shrubs that grow on dam slopes and crest may have a negative impact on certain geotechnical aspects to be avoided.

The paper shows negative vegetation impacts on small dams, which may involve geotechnical problems in the medium or long term.

1 INTRODUCTION

The environmental integration of civil works is currently an important design aspect. One of the tools available to reduce the impact of civil work projects is based on the introduction of plant species ecologically feasible. Under this approach the industry has developed a comprehensive state of revegetation treatment methods among which can be found hydro-seeding as the most extended in the civil engineering field.

In any case, the environmental integration using vegetation is feasible, provided such action does not affect the strength characteristics of structures or have a direct bearing on safety factors.

Moreover and not in a few cases, it appears that the autochthonous vegetation in the area has invaded certain parts of a public work, such slopes, surface drainage elements and in general all those places offering favorable conditions enough for plant life: water and sunlight.

The growth of vegetation in dams, however, might be problematic in contrast to environmental benefits. This paper examines several pathologies that can occur by poor maintenance of the aerial components of earth dams, when hydroseeding treatment is inappropriate or as a result of the invasion of vegetation, always in case of inadequate control or absence of maintenance activities.

2 ENVIRONMENTAL TREATMENT LIMITATIONS

2.1 *Vegetation in civil engineering*

In relation with geotechnics the effect of vegetation is a factor which aggravates the swelling and shrinkage problem of ground movements in foundation design (*Tomlinson, M.J. 1995*). The root system of isolated trees spread to a radius in the order of the height of the tree. Acording to *Tomlinson*, in sourthen England trees have caused significant drying of heavy clays soils to a depth of about 3–5 m. The same author has knowledge of several instances of heave in London Clay in the range of 100–150 mm caused by removal of mature trees, with heave movements continuing over periods of 10–20 years.

The woody vegetation growth is not a small problem in relation with geotechnical matters, but on the other hand an environmental care of the civil works has increased in the last decades. Nowadays huge projects like high speed railway, highways, ports or dams are required by law to consider environmental impact and remedial measures,

many of them with the aim of preserving from the visual impact.

With criteria based on the benefits of the existence of vegetation, several authors (*Coppin and Richards, 1990; Morgan and Rickson, 1995; López Jimeno et al, 1999*) consider vegetation and revegetation methods as an improving factor in relation with slope stabilization of linear works, reducing the volume and speed of rainwater runoff, minimizing erosion by a lamination surface but enhancing the infiltration process during rainfall periods.

Extrapolating this to the dam engineering field it seems to be a priori justified to perform project revegetation units to prevent downstream slope surface erosion like gully erosion. (Figure 1).

In the environmental engineering litterature there can be found aspects related to the benefits of revegetation treatments like hydro-seeding of plant species for other geotechnical cases. The method required for the treatment of linear works slopes is the most widespread application. In that way (*Fort López-Tello 1977; López Jimeno et al, 1999*), shows the advantages of vegetation in the sense that soil permeability and infiltration rate is greater in soils covered with vegetation than in "naked" soils. The last term seems to be against the hydraulic behaviour of the drainage system of a dam.

2.2 *Hydro-seeding over earth dam slopes*

Some small dam projects contain project units consisting on spreading a topsoil layer over the downstream slope, with hydro-seeding or hydro-mulching treatments. This environmental treatment must depend on an appropiate choice of herb species. It is quite discouraged the use of hydro-sprigging or similar treatments spreading tree and shrub seeds mixtures instead of herbs seeds on dams.

It should be noted that a "trained eye" is necessary for the inspection of dams, even more so in the case of hydro-seeding covered slopes, (*Twort et al, 1994*) specially for old earth dams, where a small surface defect may eventually prove to be related to a dangerous condition of the dam developing internally. Geotechnical visual inspections are planned, for example, to observe seepage leakage points on the downstream slope. Those points are frequently associated with the growth of greener vegetation or damp patches where there is absence of vegetation. In this last case, it is easier to find signs of irregular seepage during the inspection whenever surface erosion control measures must be taken.

Figure 1. Gully erosion on an embankment.

2.3 *Typical riparian zone vegetation*

The dams are located in streams, so it seems to be logical that vegetation growth in dams is associated to river margins. This communication does not attemp to define what type of vegetation can be found on the dams. Furthermore, the experience shows that it can be found a wide range of plant species over the same dam, in case vegetation has invaded it.

Technical information about riparian vegetation can be found in the field of soil conservation, environmental management and riverside repair and restoration. The riparian vegetation in Spain can be grouped into a comprenhensive list of typical formations (*Magdaleno Mas, 2006*).

Riparian woody species generally have shallow root systems, maybe with shallow lateral roots or deep sinker or taproots. Depending on the local conditions and vegetation existing around the dam, other species, not considered as riparian, can grow over an earth dam, like oaks or pines, with deep root systems.

In any case, trees and brush rely on an extensive root system to provide oxygen, nutrients, and moisture as they grow. The roots penetrate the embankment according to the groundwater conditions, usually without entering the saturated compacted soil, remaining above the water table.

2.4 *Root wedging of woody vegetation*

There are a number of physical weathering processes that break earth materials apart; a very common one is called root wedging. Plant roots work their way into rock mass joints. As they grow, roots create pressure on the sides of the crack enlarging it until the rock breaks apart. This is a common trouble causing slope instability in rock ground. The root wedging is a typical phenomenon in rock mass opening cracks and fissures, increasing gradually in size in a manner similar to the action of the wedges. Therefore, assuming a bad simile between dam and a rock mass it implies an

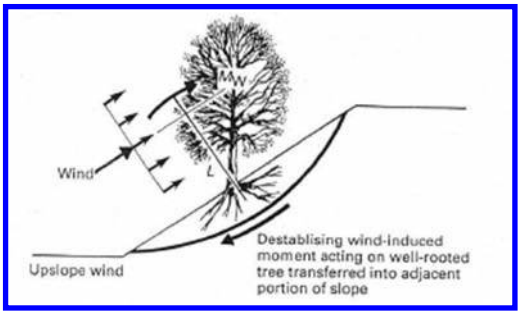

Figure 2. Upslope wind effect on a tree. (*Coppin & Richards 1990*).

unwise phenomenon in earth dams. Tree roots can force their way into a dam in the same way.

2.5 *Dam overloading caused by trees*

Another limitation is the overloading by weight and wind. If the vegetation is concentrated at the top of the slope it will increase the risk of instability. Moreover, the wind load on trees can also cause a destabilization of the slope and the trees themselves. The wind force exerted on the plant is transferred into the ground unbalancing the system of forces and inducing instability. In this way, it may cause rotation and tilt of trees, particularly if they are exposed to strong winds. Figure 2.

3 GEOTECHNICAL IMPACT OF VEGETATION ON SMALL EARTH DAMS

It is difficult to establish a list of geotechnical impacts of vegetation on earth dams because there is not enough recorded experience and no serious dam safety problems have been detected, caused by vegetation. So this paper tries to put in order some ideas about vegetation growth in relation with geotechnical behaviour of small earth dams, taking firstly into account possible benefits and continuing with theoretical hazards.

3.1 *Benefits*

Geotechnical benefits can be looked after only by the standpoint of surface erosion control, previously commented in the environmental treatment limitations paragraph.

Some investigators have documented beneficial effects of vegetation on the performance of embankments subjected to extreme loadings (*Douglas Shields, F. and Donald H. Gray, 1992*) according to the overtopping tests described by *Gilbert and Miller* (1991) which concludes that vegetation cover increased erosion resistance of the downstream slope, on river levees, by providing a protective cover and by increasing apparent cohesive strength by root reinforcement. Those studies do not consider vegetation scale (herbs, brush or trees) in this kind of natural "soil-nailing".

In that line it can even be added, why not, that roots penetration increase density of compacted soils ... But does root development need to provide nutrients (minerals) for growth by decreasing soil mass?. Or what happens when a woody plant over a dam dies or overturns?.

3.2 *Hazards*

Trees and brush represent a potential hazard when they grow and their roots easely penetrate earthen dams. Other adverse effects like erosive conditions, adverse hydraulic behaviour and obscured geotechnical dam inspections can result from the lack of maintenance activities in small dams, whose owners (Irrigation Commuinities) do not have correct information, or simply serve a residual irrigation area with possibility of abandonment.

According to the related publication of the *Commonwealth of Massachusetts—USA (Dam Owner's Guide to Plant Impact on earth dams. FEMA L-263/September 2005)*, the adverse effects can be resumed:

- Erosion Impacts: Mainly internal erosion (piping) provoqued by the decaying roots of dying woody vegetation that creates a seepage path.
- Woody vegetation shades the embankment and reduce dense grass coverage, which can be useful to prevent gully erosion.
- Trees and brush loosen compacted soils of the earth dam or clay core in other case.
- Roots can penetrate existing cracks and joints in the foundation rock.

There can be observed more hazards in relation with vegetation and dams:

- Overturning and uprooted trees provoque holes on the dam.
- Spillways capacity could be reduced when falling trees mixed with brush get stuck into the water flow cross-section.
- Clogging drainage systems.
- Uplifting and displacing concrete facilities.
- Providing cover for burrowing animals.
- Opening cracks in foundation or abutment joints between dam and natural ground.
- Local slope stability of intermediate berms on downstream face.
- Interfering with effective dam safety visual monitoring.

The problem seems to be clear once the growth of woody vegetation on the dam's body has been allowed, by neglecting maintenance activities. According to the document drafted by the *Committee on the Safety of Existing Dams, Water Science and Technology Board (USA) entitled "Safety of Existing Dams. Evaluation and Improvement. 1983"*, in operating dams it is a requirement to inspect and assess the problems with trees and shrubs that grow over these dams. Cited in the text, as a rule, shrubs and trees should be removed and its growth prevented in the body of the dam.

4 SOME PATHOLOGICAL CASES

4.1 *Woody vegetation on dam crest and upstream slope*

It has been observed that trees and brush use to grow in alignments, 1–2 m over the saturation line in Full Supply Level, when reservoir presents high water level almost all the year.

There can be observed two different examples of invasive vegetation on small earth and rockfill dams with clay core, over the upstream slope (Figure 3) and over the crest. (Figure 4).

Old rockfill dams can present a top part of the dam, near the crest, generally clogged with soil from crest pavement and sediments. Trees grow through the external rockfill layer.

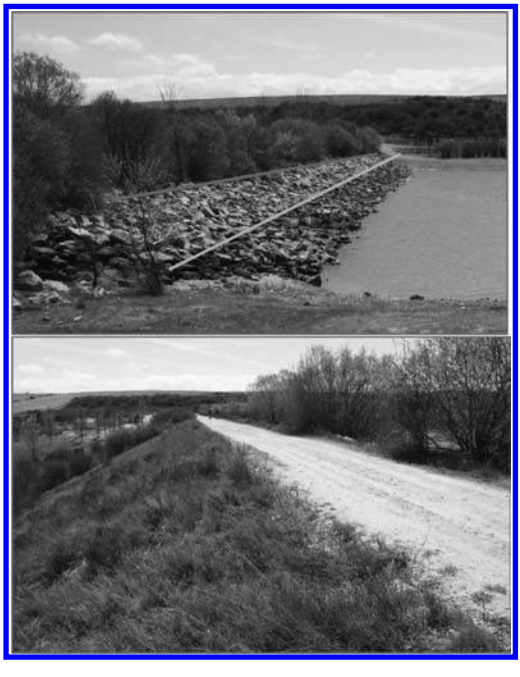

Figure 3. Tree line (red) over earth dam's maximum normal saturation level (blue). Image taken from crest, below. (L.G. CEDEX)

Figure 4. Tree line (red) over rockfill dam's maximum normal saturation level (blue). (L.G. CEDEX)

4.2 *Woody vegetation near earth dam concrete elements and drainage net*

Although earth dams are made of compacted soil and rockfill layers, lateral spilways and drainage elements are concrete formed. Trees growing in soil behind walls, near water channels and other structures are able to push walls, uplift channel foundations and impede the free flow of water.

Figure 5 shows how trees have accumulated behind a wall and figure 6, can be of interest, where brush is the backfill of an energy dissipator channel at the low-level outlet exit of an earth dam.

It could be difficult for vegetation to grow enough to break concrete structures, but the problem in relation with maintenance is important in two different ways:

- Maintenance labours and planned dam inspections are significantly affected or disabled in the worst case.
- Vegetation invades drainage elements reducing the free water flow cross section (Figure 7). In other case roots uplift the channel reducing or reversing the angle of descent, forming puddles (Figure 8). The potential problem has to do with the eventual overflow of water.

Spillways must be always clean of stones and tree trunks which can prevent the correct water flow in normal or emergency situations (floods). The spillway is an important element of the dam, so that vegetation has to be removed at the first sight of growth. Situations provoqued by a bad maintenance as shown by the next cases have to be erradicate:

1. Trees growing on the resorvoir near the spillway lip are water obstacles (Figure 9), and potential clogging bases. Died vegetation floating or plastic elements washed away during a storm

Figure 5. Trees behind a reinforced concrete wall, which is an abutment of the spillway channel bridge. (L.G. CEDEX)

Figure 6. Brush behind an energy dissipator on deep outlet exit. (L.G. CEDEX)

Figure 7. Drainage net element of a downstream berm reducing cross-section of water flow. (L.G. CEDEX)

Figure 8. Trees rooted behind a service surplus channel downstream of a earth dam. (L.G. CEDEX)

Figure 9. Small tree growing near the spillway static crest of a dam. (L.G. CEDEX)

can get stuck affecting the hydraulic behaviour of the spillway.

2. The spillway channel of an earth dam is generally concrete formed over the upstream and downstream sides of the dam. But there are cases in which the concrete channel is followed by a rip-rap cover channel. Figure 10 shows a 4–5 m height tree in the middle of the section change of a spillway channel, just below the dam crest.

The mentioned rip-rap channel is completely invaded by trees as it can be seen in Figure 11.

Other example is shown in Figure 12, in the case of a large spillway channel whose end coincides with 5–6 m height trees.

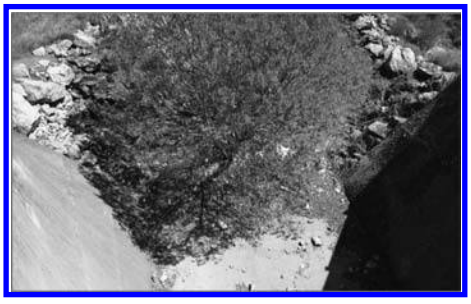

Figure 10. Big tree growing just below the dam crest in the spillway channel of an earth dam. The channel continues with a rip-rap cover along the downstream slope. (L.G. CEDEX)

Figure 11. Trees invassion of a rip-rap covered channel through the downstream slope boulder of an earth dam. (L.G. CEDEX)

Figure 12. Trees at the end of a spillway channel. (L.G.CEDEX)

5 SMALL DAM VEGETATION BACKGROUND

Woody vegetation root penetration is not a beneficial effect on the performance of earthen dams, it does not stabilize a soil mass, but probably loosens the soil of an embankment slope and creates a condition more conducive to surface water penetration and slope failure (*FEMA 534/September 2005*). Dense growths of trees and woody vegetation not only present a hinderer to proper dam safety inspection, but also are detrimental to the structural integrity of small earthen dam.

The results of one survey, by the International Commission of Large Dams (ICOLD), were reported in its publication "Lessons from Dam Incidents" (*USA N.J. Snichtter, 1979*). This document contains a table, reproduced here as Figure 13, in which it is evident that the failure risk decrease with height increasing, and embankment (fill) dams with a height between 15 and 20 m represent 50 percent of failures, as it is remarked in red on the same figure.

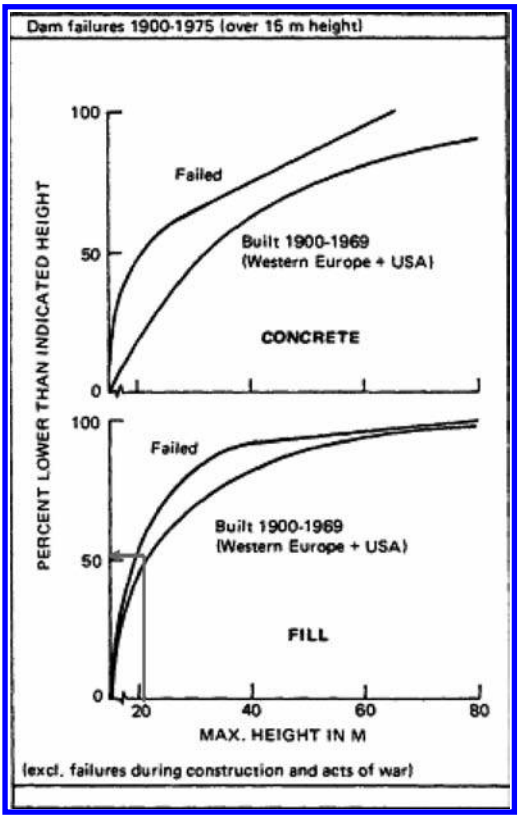

Figure 13. Dam failure (ICOLD). Sample form 1900–1975, excluded failures during construction and acts of war.

This figure does not show correlations between failure modes and causes but it seems of particular interest to see that small dams are less safe.

6 SOLUTIONS AND RECOMMENDATIONS

6.1 *Vegetation treatment for surface erosion protection*

Surface runoff erosion is one of the most common maintenance problems of embankment structures. In bald areas or areas where the protective cover is sparse embankments are more susceptible to surface runoff erosion problems. If not corrected, this erosion can become a more serious problem.

If it is expected that surface erosion or rainfall form gullies on the downstream slope some dam owners determine the planting of some type of vegetative cover (usually grass) on the slopes to provide erosion protection. The root system of the vegetative cover holds the surface soil in place and protects the slopes from wind and surface runoff erosion above and below downstream slope. In this case a careful maintenance of the plants should be considered.

US FEMA consider that the grass cover should be maintained to a maximum height of approximately 10–15 cm to allow proper embankment inspection. In addition, well-maintained grass helps prevent animal burrowing and controls deep-rooted vegetation.

Sometimes it is preferable to protect downstream face with a rip-rap cover or with other appropriate geotechnical treatment.

6.2 *Woody vegetation affecting the dam*

The best situation for a geotechnical dam inspection is the "naked" slope. This helps situation for monitoring and maintenance, identification of dam patches, caused by eventual seepage, cracking, sinkholes, slumping, settlement deflection, and other signs of pathology.

Although a surface protection, is desirable in arid or semi-arid regions, where gullies can be formed by strong rainfall, the growth of free vegetation may cause undesirable deep-rooted vegetation, such as large shrubs and trees, when maintenance is a secondary fact.

Large trees could be blown over and uprooted during a storm. The resulting large hole left by the root system could breach the dam or shorten the seepage path and initiate piping. Root systems associated with deep-rooted vegetation develop and penetrate into the dam's cross section.

When the vegetation dies, the decaying root system can provide paths for seepage and cause piping to occur. Even healthy root systems of large vegetation can pose a threat by providing seepage paths. These seepage paths eventually can lead to internal erosion and threaten the integrity of the embankment.

Trees and shrubs are undesirably growing on or adjacent to embankment dams. The location, size, type of tree, and prevailing policy will determine the course of action at a given site.

The best approach to trees in the crest, slopes, and adjacent to the dam is to cut them down before they reach significant size. If large trees have been cut down, but the root system is not removed, the area around the remaining stumps must be carefully monitored for signs of seepage.

It can be understood, as a basic rule, first of all not allowing to grow trees, but if there are already trees on the dam it has to be considered case by case.

One solution could be cutting the trees to ground level, leaving stumps and roots particulary if it is ryparian shallow rooted vegetation and there is not seepage damage detected. In this case it must be planned a yearly maintenance because there is high reproducing probability. It could take years to "kill" some tree species by planned cutting. It is not recommended to use herbicidal deliveries when treating the upstream face. Finally a programmed inspection routine has to be set up with particular emphasis on this dam area.

There are approximate methods to estimate the extent of the root system of trees depending on the species and other aspects to be evaluated by technical experts in vegetation. Some are explained by *Coppin and Richards* (*1990*). If a serious problem is detected it would be required to do a special study by a multidisciplinary qualified staff of enginners.

6.3 *Animal damege associated with vegetation*

Burrowing animals make nests and passageways. These passageways may cause internal erosion failures if they connect the reservoir to the downstream slope or shorten seepage pathways through the dam.

This kind of animals should be avoided from dams.

6.4 *New earth dam projects*

To improve the long term behaviour of a dam it must be considered to effect an optimum compaction of soils. Densely compacted soils discourage root elongation as a result of exclusion of most of the requirements for healthy tree growth (*FEMA 534, 2005*).

7 CONCLUSIONS

It is strongly recommended to keep the earth dams cleared of vegetation by the owners.

The public administration should plan programmed inspections in order to check that a good maintenance activity is being carried out by the owners.

Vegetation could be a big trouble for small dams.

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Extraordinary investments

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Rehabilitation of dams in South Africa…. 40 years on

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ABSTRACT: An overview is given of South African dam statistics of dams built and owned by the Department of Water Affairs and those rehabilitated during the past decade. The rehabilitation statistics will include reasons for the rehabilitation and will highlight the effect of changing flood estimates as well as dam safety standards. The changes in decision making process and prioritisation criteria during the past 40 years are discussed briefly. Department of Water Affairs (DWA) owns the majority of larger dams in the South Africa. The discussion focuses on the processes developed and followed by DWA to decide when to rehabilitate dams against the light of various external factors. The emphasis is on the probabilistic methodology introduced in the mid 1980's and more specific the present studies to improve the methodology. A few case histories from the various "rehabilitation eras" are presented briefly to illustrate the differences of approach followed during the past 40 years.

1 INTRODUCTION

The Department of Water Affairs (DWA) of South Africa owns 359 dams of which 218 are listed in the ICOLD World Register of Dams. The majority of these dams was designed and constructed by DWA itself. The number of DWA owned and ICOLD listed large dams built per decade as well as the different types of dam are represented in Figure 1. It clearly shows that dam building by DWA reached its peak during the eighties with a steep decline towards the end of the previous century. Dam engineering therefore is experiencing a shift from the design and construction of dams (using mainly design criteria), to maintenance and rehabilitation of dams (using failure criteria).

It is also important to note that the construction of concrete dams peaked in the 1960's and 1970's while construction of earthfill dams peaked during the 1980's.

A large number of these dams do not comply with current design criteria. The majority of these involve either insufficient spillway capacity or mechanical equipment in need of refurbishment. For example 52 dams have been prioritized for rehabilitation in the period since 2003. 60% of them do not have sufficient spillway capacity while 15% require major refurbishment of their mechanical equipment. The problems with insufficient spillway capacity are mainly due to not having long enough hydrological records during the design of the dam. The development of an empirical method for extreme flood peaks using a Franco-Rodier based methodology of historically observed flood peaks (Kovacs 1988) have however gone a long way to mitigate this problem.

2 DWA'S RISK BASED DECISION MODEL AND METHODOLOGY

The decision making process used for the rehabilitation of dams owned by DWA is directly related to the development of dam safety processes in South Africa. Two major dam related incidents in Europe in the 1950's and 1960's (the failure of Malpasset Dam in France in 1959 and the slope failure into Vaiont Dam in Italy in 1963) that resulted in huge human and financial losses drew attention to dam safety in Europe in the 1970's. In South Africa only the odd dam safety inspection took place during this period.

In 1977 a decision was taken to establish a dam safety program in South Africa. This culminated in the promulgation of dam safety regulations for all dams with a safety risk in 1986 (RSA 1986) as well

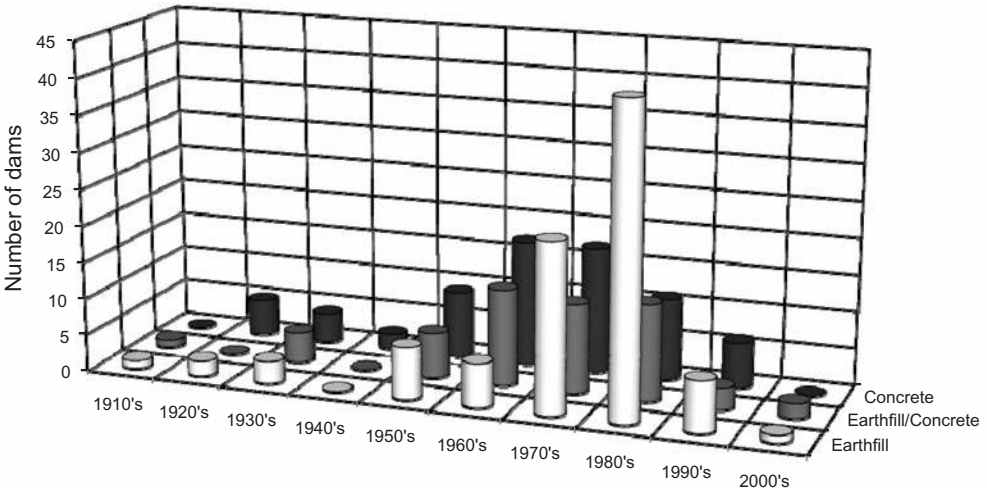

Figure 1. Number and type of DWA owned listed by ICOLD built per decade.

as the development of a risk based decision model and methodology for the probabilistic safety evaluation of DWA owned dams (Oosthuizen 1987). The methodology for the risk-based dam safety management of dams owned by the Department of Water Affairs was formally approved in 1987 by DWA's management.

This methodology has since been continuously updated and refined (Oosthuizen et al 1991, Oosthuizen & Elges 1998, Oosthuizen 2000, Oosthuizen et al. 2002, Oosthuizen & Hattingh 2006, Oosthuizen & Hattingh 2007, Hattingh & Oosthuizen 2009 and Oosthuizen 2009). Some of the important aspects of the methodology are discussed in more detail below.

Although risk analyses are not required by South African dam safety legislation, 'Level-0' risk analyses have become a standard part of all compulsory 5 yearly dam safety assessments conducted by the DWA since the late 1980's. A 'Level-0' analysis is basically a fast desktop study to assess the "risk profile" of a dam. Higher levels of analyses are only performed when the impact or risk criteria warrant better estimates of the probabilities of failure. Rehabilitation work to satisfy acceptable failure criteria are in most cases expensive. Normally a limited annual budget is available to perform rehabilitation. One of the tasks for the dam safety engineers therefore is to avoid unnecessary cost and to optimise the results from the limited rehabilitation budget. In most instances the financial gain (reduction in risk cost) is less than the cost of remedial works.

The methodology for the prioritisation of remedial works uses the results of the risk-based decision model and methodology and can be summarised in five distinct steps as follows:

- Determine the probability of a dam failure (break)
- Perform a dam break analysis;
- Assess the potential losses in the event of a dam break and evaluate the impacts and risk levels;
- Determine priorities using a multi criteria decision-making model; and
- Present the facts.

2.1 *Main concepts*

Some of the main concepts of the risk-based decision model are:

- *Loss of life* is treated as an impact and expressed as fatalities per exposed hour to account for residents in the flood path. The majority of risk methodologies make use of the so-called single line F/N graphs (Fatalities versus Population at Risk (PAR)) to represent life loss, which also serves as acceptability criteria. The number of persons exposed to the dambreak flood and the likelihood of its occurrence are the governing factors determining the dreadfulness of this event and consequently the relative impact on those affected. This aspect is addressed as a separate impact;
- *Financial losses* due to a dam failure comprise direct and indirect financial losses. It must be noted from available statistics on fatal traffic accidents in South Africa that only 10% of the total calculated financial loss of all types of fatal traffic accidents in 1998 was direct costs—the rest was indirect costs;
- *Direct losses* are for example, damage to infrastructure, loss of improvements, crops, etc. as well as the cost of emergency relief. Certain direct

financial losses cause a lot of confusion, for example, loss of future benefits and the replacement cost of the dam. From the time a dam breaks the future benefits are lost. The value of that stream of benefits is equivalent to the "value of the dam". Rebuilding or even repairing a breached dam is a new project with its own costs and benefits. Therefore either replacement costs or loss of future benefits should be included in the loss, but not both. Preference is given to the second approach;

- *Indirect financial* losses (not restricted to the inundated area only) are for example loss of future benefits. Monetary values are not attached to human life only loss of their future earnings is included under indirect costs;
- *Socio-economic importance* is quantified subjectively. The gross geographical product of the area, (i.e. the total income or payment received by the following production sectors: agriculture, mining and manufacturing) as well as the gross geographic product per capita is used to guide the dam safety assessor with the classification of the relative socio-economic importance;
- *The social importance* is classified subjectively. Population density of the area, availability of water per capita and alternative sources of water are used to guide the choice of the relative social importance;
- *Ecological impact* is even more difficult to quantify. A group of environmentalists of DWA have studied typical stretches of rivers in order to categorise stretches of all the rivers into 6 classes. The quality and quantity of water as well as the influence on bio mass were some of the aspects they have taken into account. This is used to assist with the classification of the relative environmental importance; and
- The *Risk level* of a dam is quantified by comparing the risk to human lives exposed to the particular hazard with the annual risk of financial losses (both direct as well as indirect). The dividing line between acceptable and unacceptable corresponds with the level of risk accepted voluntarily in South Africa and has been determined from the statistics of construction accidents as well as those of all modes of transport in South Africa.

The results of these (impacts as well as risk level) are presented by a set of graphs with the relative importance or quantity represented on the X-axis and the probability of occurrence on the Y-axis. The results of the assessment are presented in reports as six graphs on a single page. These graphs are used mainly to convey information for comparison and decision-making.

2.2 *Unique features*

The basic methodology is unique in at least a few aspects:

- Born out of a need for a meaningful and justifiable method to allocate scarce funds, in other words to prioritise remedial works on a probability basis and not to continuously motivate for funding for remedial works on an ad hoc basis;
- Depending on the situation the methodology makes provision for various levels of analyses, starting from a desk study (level 0) and progressing through various levels of sophistication as deemed necessary;
- The use of simple yet rigorous methods of analyses (e.g. Failure domains, Point Estimate Methods, Coefficients of Variation etc.);
- Truncated probability distributions (mainly Beta distributions) are used for more rigorous analyses (after validation of the shape of the distribution);
- The basic metrology principles of monitoring are applied (such as the validation of measurements/observations, determination of the standard error and confidence intervals);
- Emphasis is placed on the confidence levels of the results and it is clearly indicated in the presentation of the results;
- Presentation of the results by means of a series of graphs, which provide guidelines (not hard and fast rules) to the decision-makers on the acceptability of risk.

3 PRACTICAL PROBLEMS ASSOCIATED WITH RISK ANALYSES

3.1 *Failure modes*

Calculation of the probability of failure is the most important component for the evaluation of the safety of a dam as impact and risk is directly "related" to probability of occurrence of the failure. Analysing South African dam incidents indicated that:

- overtopping is the cause of more than 50% of South African dam incidents;
- followed by internal erosion (mainly due to dispersive soils) with 30%;
- slope instability only accounts for a mere 10% of dam incidents; and finally
- other modes of failure such as human error, unacceptable engineering practices, liquefaction or failure of appurtenant structures adds up to 25%.

(Note the percentages do not add up to 100% because some of the incidents had multiple causes). Relative probabilities can be employed to account for those modes of failure that have not been calculated.

3.2 *Probability of failure*

The practical limit of calculated probabilities of failure is by default 1E-6. In practice it means that a well-engineered dam with no known deficiencies has a probability of failure of 1E-5 to 1E-6 by default. For the sake of the inexperienced reader the following basics will be described in non-technical language:

- Probability varies between one and zero (1 and 0). The value is 1 when it is a fait de compli (e.g. the failure of a dam) and zero when it is practically impossible;
- Probabilities can be time related (i.e. per annum) or absolute (i.e. per "life time"). It is customary in risk analyses to convert all probabilities to per annum values in order to compare apples with apples (and to be able to "add" them together when required). The authors favour the exponential distribution to calculate annual probabilities of events without a time base such as slope instability (Oosthuizen et al: 1991);
- Probability distribution refers to the estimated path that the values would follow between 1 and 0;
- Extreme events such as floods and earthquakes do not follow a linear distribution between 1 and 0 and their path can be described better by means of a logarithmic scale;
- Annual probability of extreme events (e.g. floods, failures etc) is therefore as a rule expressed as on a logarithmic scale (for example 0.1, 0.01 and 0.001 are expressed as 10^{-1}, 10^{-2} and 10^{-3} or in another so-called engineering notation 1E-01, 1E-02 and 1E-03); and
- Return Period is another way to express these annual probabilities and it can be approximated by the inverse of the probability, e.g. 1 in 10, 1 in 100 and 1 in 1000 years in the example above. The use of Return Period is not favoured by many engineers as it gives a false impression that the event can only occur once every so many years, e.g once in 10, 100 or 1000 years in the example above.

3.3 *Loads and response*

The following loads, resistance and response aspects are singled out for further elaboration:

- Floods are the main source of uncertainty and the major thrust of the analyses should therefore be in the field of flood hydrology;
- Internal erosion (piping) failure has for various reasons only been evaluated subjectively. The main reason for these failures is either inadequate protection of dispersive soils by properly designed filters or damaged outlet pipes through fill material.
- Dynamic loads are usually of secondary importance at dams in South Africa. Apart from the failure of a small farm dam and damage to the old Ceres Dam, the author is not aware of any other incidents. Complex analyses can seldom be justified due to uncertainties with regard to the seismic loads and dynamic material properties. In the absence of any measured seismic data (time history) at or near the dam, the dynamic loads can be approximated with a scaled version of the 1940 El Centro earthquake (either the time history or the response spectrum). This specific El Centro earthquake is preferred due to its wide frequency range. The dynamic response (modes and frequency) of existing dams can be determined using Ambient Vibration Measurements (AVM). These measurements are relatively simple to perform (the equipment and associated software have to be quite advanced though). These 3-D AVM analyses provide sufficient information to calibrate finite element models for subsequent static and dynamic analyses.

4 REHABILITATION ERAS

Rehabilitation of dams and its accompanying decision making process and prioritisation criteria used during the past 40 years can be divided into four distinct eras, the majority of which is directly related to the development and implementation of the risk based decision making methodology:

- *The early "ad hoc" years (before 1987)*—before the implementation a systematic safety management programme and of the risk-based decision making methodology, remedial work were done on an ad hoc basis. This only followed on either an incident happening at a particular dam or the occasional ad hoc dam safety evaluation;
- *The "lean years" (1987 to 1994)*—During this era the implementation of the risk based decision making coincided with an acute shortage in public funding (due to a failing economy and international sanctions).
- *The imminent failure years (1987 to 2003)*—A period followed after the first democratic elections in 1994 during which all the former Bantustan homeland dams were taken over by DWA. A significant amount of these dams were not designed and constructed to present day standards (due to a variety of reasons). This gave rise to emergency rehabilitation being done at a number of dams to stop imminent failures; and
- *The state of the art" years (2003 to current)*—after years of underfunding for maintenance as well as rehabilitation, funds were made available as part of an extended public works

programme to try and satisfy current dam safety requirements using the results of the risk based dam safety evaluation methodology as motivation. It is important to note that this era to a large extent did not entail significant prioritization at individual dams once a decision was made to rehabilitate these dams as sufficient funding has been available.

In the following sections case studies for each of these eras are described in more detail.

4.1 *Vaal Dam—the early "ad hoc" years (before 1987)*

Vaal Dam, with a capacity of approximately 2535 million m^3, is situated approximately 56 km south of Johannesburg on the Vaal River. The dam is considered to be the most strategic water retaining structure in South Africa. Not only is it the major source of water for Gauteng province (the economic heart land of South Africa), but it also provides water to major industries in the Mpumalanga Province and stabilizes the flow in the Vaal River for consumers downstream. DWA undertook the design and construction of the original dam, which was completed in 1938. The original dam wall comprised of a short concrete gravity non-overspill section on the left flank, a central concrete gravity spillway, a main earth embankment and a saddle embankment on the right flank.

In order to meet the increasing demand for water from the dam it was raised in the early 1950's. The concrete spillway section was raised with 3 m, the main earth embankment and saddle embankment was raised and lengthened and 3 m high gates were provided on top of the raised concrete spillway section.

Stability analyses conducted in 1975 however indicated that the structure did not comply with the dam safety standards of the time and the need for rehabilitation work was identified, specifically in so far the spillway capacity and the stability of the concrete gravity sections were concerned. The intended rehabilitation work was also seen as an opportunity to increase the capacity of the reservoir. In the 1980's the gates were replaced with 6.35 m high gates and the bridge deck, non-overspill concrete section and the earth embankment were raised, increasing the wall height to a maximum of 63 m above lowest foundation level. The dam wall was stabilized using 88 pre-stressed cables and by increasing the cross sectional area of the concrete sections of the wall. The cables stretched from the non-overspill crest to a maximum depth of 30 m below the foundation level. In addition, a 680 m long tunnel was excavated 20 to 30 m below the foundation level over the full length of the concrete sections of the wall to relief up-lift pressure. See Figure 2 for a typical cross section through the concrete gravity section and Figure 3 for an aerial view of the current dam.

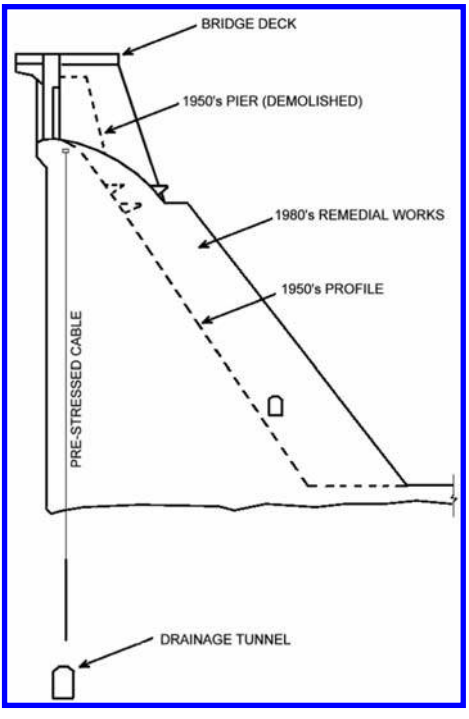

Figure 2. Typical section of the concrete gravity section of Vaal Dam showing all the remedial works.

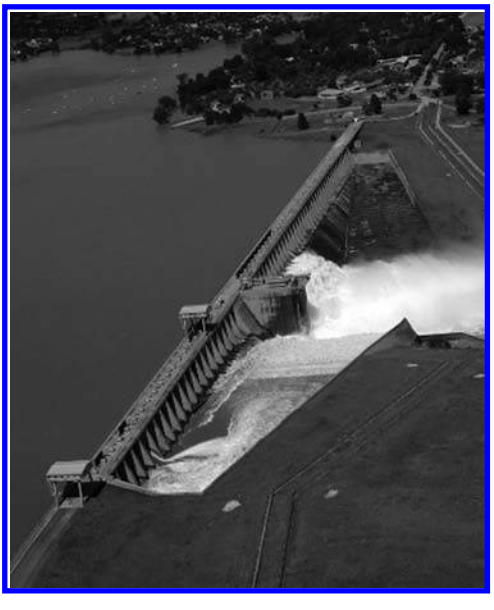

Figure 3. Aerial view of Vaal Dam.

4.2 *Calitzdorp Dam—the "lean years" (1987 to 1994)*

Calitzdorp Dam is a 31.7 m high concrete gravity dam located just north of Calitzdorp in the Western Cape. At the time of construction (it was completed in 1918), it was the highest concrete dam in South Africa. It is mainly used for irrigation and domestic water supply.

During the first round of compulsory dam safety evaluations of DWA dams in the late 1980's, Calitzdorp was one of those that did not satisfy dam safety requirements (the drainage system of the concrete structure was not functional and the spillway capacity was not sufficient). Using the risk based prioritization methodology with the results of the first round of risk-based dam safety evaluations, it became apparent that remedial works at Calitzdorp Dam provided one of the best value for money options. A decision was made to proceed with the remedial work based on the results of the prioritization methodology. The remedial work was done in the early 1990's and included improving the spillway capacity as well as the internal drainage system to decrease the influence of uplift. See Figures 5 and 6 for before and after photos of the dam.

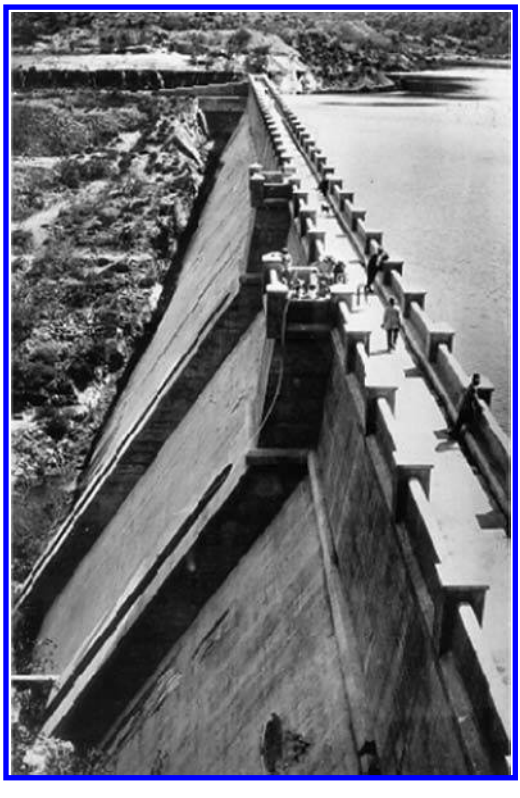

Figure 4. Calitzdorp Dam before rehabilitation.

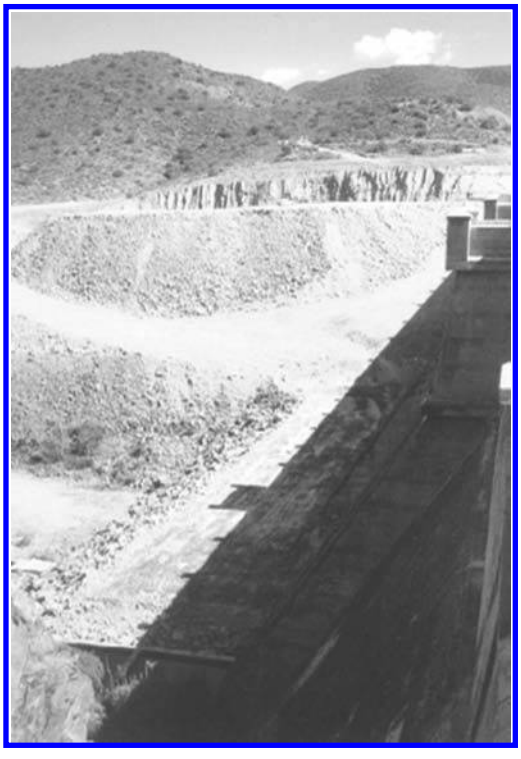

Figure 5. Calitzdorp Dam after rehabilitation.

4.3 *Thapane Dam—the "imminent failure" years (1994 to 2003)*

Thapane Dam is located on the Makobodi River near Tzaneen in the Limpopo Province of South Africa. In February 2000 severe flooding was experienced in this part of the country and the 14 m high earth embankment of the dam was overtopped, causing severe erosion on the downstream face (see Figure 6). The erosion gullies on the downstream side, some of which were up to 4 m in depth, posed an immediate threat to the safety of the dam. Anticipating more heavy rainfall in the area, DWA's construction unit raised the crest of the embankment with 0.5 m as a precautionary measure. This action prevented an imminent failure, as only days later the dam wall was subjected to a second flood, with the peak flood level rising to just below the raised crest.

An evaluation of the capacity of the existing bywash type spillway indicated that it was totally inadequate in terms of dam safety standards and it was decided to increase the capacity to an acceptable level. This was done by raising the non-overflow crest with 3 m and changing the existing bywash type spillway to a side channel type spillway. A road bridge was also constructed over the new spillway as part of the remedial works. The earthworks on

Figure 6. Thapane Dam after severe overtopping in February 2000.

Figure 7. The rehabilitated Thapane Dam.

the embankment were completed in July 2001 and the work on the spillway was completed in July 2003 (see Figure 7 for the rehabilitated dam).

4.4 *Lakeside Dam—the "state of the art" years (2003 to current)*

Lakeside Dam is a 8 m high earthfill embankment with 36 Mirza gates. The dam is located on the Mooi-river just upstream of Potchefstroom in the North-West Province. Almost a century after the dam was completed in 1909, it was identified for rehabilitation due to dam safety deficiencies (insufficient spillway capacity and mechanical equipment that needed major refurbishment).

The following work was done at the dam:

- Raising and widening of the dam embankment crest to prevent the dam from overtopping during the occurrence of floods when the gates are closed. It is interesting to note that the spillway capacity needed to be increased from 276 m^3/s to 782 m^3/s (see Figure 8);
- Provision of a toe drainage system on the embankment, to stop undermining of the earthfill embankment and possible failure;
- Refurbishment of the 36 Mirza gates which were originally imported from England in 1905. These are unique as they have a balancing system that automatically opens the gates when required. As this was one of a few remaining dams in South Africa with Mirza gates, it was decided to refurbish the gates instead of removing it from the spillway. Three gates are currently being modified to open and close manually should the need arise (see Figure 9);
- Other maintenance work like provision of a concrete apron slab in the spillway return channel to control weeds; and
- Creating a paved walkway on the crest of the dam embankment.

Figure 8. The earthfill embankment of Lakeside Dam after rehabilitation.

Figure 9. The rehabilitated Mirza gates of Lakeside Dam.

Construction for the rehabilitation of the dam started in August 2008 and the project was successfully completed in October 2009.

5 IMPROVEMENT OF RISK-BASED DECISION MODEL

A process is underway to improve the methodology of the risk-based decision model as well as the presentation of the results. The graphs used for the results of the level 0 analyses have recently been simplified into a "Sudoku" format (see Figure 10).

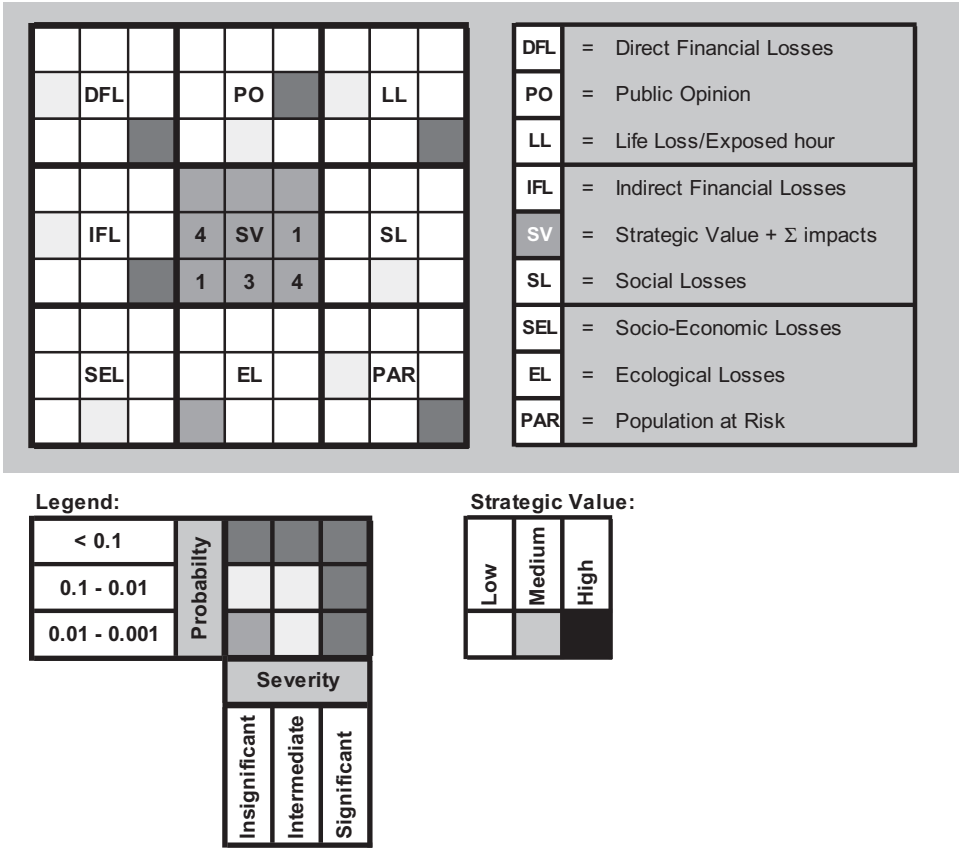

Figure 10. The proposed "Sudoku" format for presenting the results of the risk-based decision system.

In addition two more impacts have been added recently, namely Public Opinion and Strategic Value and the Risk Level have been omitted (mainly because a monetary value is indirectly assigned to a human life). The risk level is basically replaced with the strategic value and the block receives prominence with a different (grey scale) colour scheme to indicate the relative importance.

The vertical axis of all 9 graphs are probability of the particular hazard (which, in most cases is the probability of failure but not referred to as such in the graph as human errors may cause the hazard at another probability and hazard).

The severity of the hazard is represented on the horizontal axis of the 9 graphs. The impacts are represented as coloured blocks on the various graphs. The impacts are indicated in the centre block which also represents the Strategic Value. For probabilities less than 0.001 or for insignificant losses/hazard the blocks will remain white.

6 CONCLUDING REMARKS

The majority of DWA owned dams that does not satisfy current dam safety requirements, either does not have sufficient spillway capacity or need major refurbishment of their mechanical equipment. Insufficient spillway capacity is mainly as a result of not having a long enough hydrological record during design and is especially relevant for earth- and rockfill dams.

The prioritization methodology that uses the results of the risk-based decision model has proved to be an effective tool to optimize the use of funds available for rehabilitation, especially when only limited funds are available.

Risk assessment is not an exact science. Probabilistic analyses are dependent on consistent sound engineering judgement. The outcome can be biased by the preferences of the engineer(s) performing these analyses and peer review is essential. Care must be taken to reveal limitations and

not to claim more than the method can provide. The limitations of extreme probability distributions on the outcome of risk analyses should not be overlooked. Truncated probabilities should preferably be used.

Risk analyses should be performed step-wise, always starting at the lowest level. An analysis stops when the desired degree of refinement has been achieved or when uncertainties in the input data make further refinement not cost-effective or just plain meaningless. It seldom warrants exploring each element of the analysis to its state-of-the-art.

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Review of condition surveying works of civil structures and reservoir area for rehabilitation of the Piva hydro power plant

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ABSTRACT: Testing of Piva HPP civil structures, within the Project of Rehabilitation and Modernization, was conducted through three segments: Testing of Civil Structures, Testing of Reservoir Sedimentation and Storage Capacity and Testing of Unstable Areas and Grout Curtain Conditions. Performed tests established the actual state of HPP Piva structures and reservoir and enabled defining the scope of necessary rehabilitation works, in order to provide a new working cycle of the rehabilitated equipment and facilities, in the following decades.

1 INTRODUCTION

The Piva Hydro Power Plant (HPP) is a storage type plant, located in the north-west part of Montenegro on the Piva river, approx. 10 km upstream of the place where the Tara and the Piva rivers form the Drina river. The road Niksic—Foča crosses over the dam crest.

The Piva HPP is a peak hydro power plant with an annual electric power generation of around 744 GWh. It has been engaged in regular electricity generation since March 1976.

A 220 m high concrete arch dam of the Piva HPP forms 48 km long reservoir with capacity of 880×10^6 m^3.

Intake structures and tunnels for transporting water to the underground powerhouse and its discharging through the lower surge tank into the river bed (at approximately 200 m downstream of the dam) are located in the left dam abutment. The switchyard is also installed underground.

After more than 30 years of its operation, the Electric Power Company of Montenegro (EPCG) has received a credit from the Kreditanstalt für Wiederaufbau (KfW), to be used for the Piva HPP Rehabilitation Project. This Project comprises rehabilitation of the equipment and plant structures, which should enable further 30 years of HPP's safe, efficient and reliable operation.

The Piva HPP Rehabilitation and Modernization Project are divided in two Phases. During the Phase I—Project Preparatory Phase, and based on getting familiar with the problems noticed during the HPP's operation up to now, necessary testing and surveying of the present condition of structures and equipment were contracted and performed. The objective of these tests is a reliable defining the type and scope of necessary rehabilitation works, which would be implemented during the Phase II of the Rehabilitation Project. The planned and executed condition surveying

Figure 1. Location of the HPP Piva.

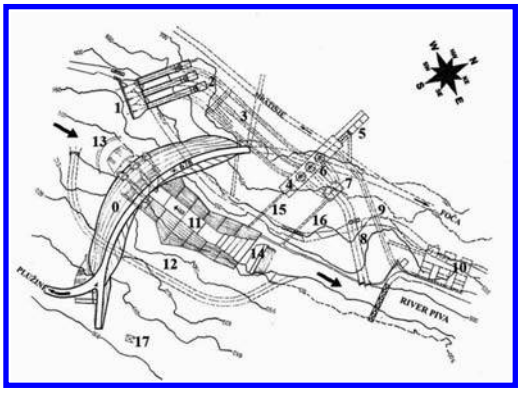

Figure 2. Layout of the Piva HPP: 0. Arch dam; 1. Intake structure; 2. Valve chambers; 3. Headrace tunnels; 4. Powerhouse; 5. Transformers; 6. Draft tubes; 7. Lower surge tank; 8. Tailrace tunnel; 9. Access tunnel; 10. Switchyard; 11. Stilling basin; 12. Diversion tunnel; 13. Upstream cofferdam; 14. Downstream cofferdam; 15.–16. Appurtenant galleries; 17. Monitoring station.

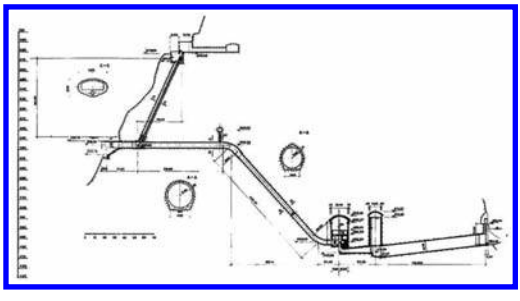

Figure 3. Longitudinal section through the underground part of the Piva HPP.

works of the Piva HPP civil structures and reservoir area will be presented in this paper.

The main problems registered at the Piva HPP during its operation period refer to leakages through dam body and concrete lining of the tunnels and caverns, failure in operation of certain number of monitoring instruments, large rockfalls from the slopes downstream of the dam, high groundwater levels at downstream piezometers and landslides at certain locations along the reservoir area.

Maintenance of civil structures during the previous period was aimed, in the first place, at execution of geotechnical works related to reparation of the noticed leakages. Data related to performed rehabilitation works and accomplished effects were not systematically sorted out. Some parts of the structures were inaccessible for the regular annual inspection works, therefore it was not possible to make a complete insight into condition of the Piva HPP civil structures.

Figure 4. Protective structure above the dam crest.

Problem of rock falls downstream of the dam, so far has been resolved by construction of protective structures, such as, for example, a strong protective structure at the left abutment, at dam crest elevation, formed by the anchors and concrete grid, the protective reinforced-concrete structure at the entrance into underground part of the plant or covered metal bridges at the entrance into the galleries in the dam body.

Also, the User did not possess the reliable data related to reservoir storage capacity, to be used as the basis for the reservoir sedimentation analysis.

Problems of landslides in the reservoir area, caused by construction of the dam were resolved by compensation of the damage to the owners of the properties at which ground movements were recorded. This refers in the first place to the largest landslide recorded in the zone of the Mratinjski stream.

Analysis of landslides impact to the reservoir storage capacity and possible measures for their stabilization have not been analyzed during the previous operation period.

Based on the Tender Document prepared by the Consulting Consortium constituted by Colenco Power Engineering AG, Baden, Switzerland and Energoprojekt-Hidroinženjering Co. Ltd., Belgrade, Serbia, the condition surveying works were contracted and performed through three independent segments:

1. Testing of Civil Structures,
2. Testing of Reservoir Sedimentation and Storage Capacity,
3. Testing of Unstable Areas and Grout Curtain Conditions.

At the International Tender which was invited at the end of 2009, the job was awarded to the companies and institutes specialized for this type

of works, from the neighbouring countries—Croatia, Serbia and Montenegro.

Based on the condition surveying results, the Preliminary Design and Feasibility Study for Rehabilitation of the Piva HPP will be prepared.

2 SCOPE OF CONTRACTED TESTING WORKS

2.1 *Testing of civil structures*

The following testing works were envisaged in the Tender Document:

- Visual inspection
- Interior condition survey (testing) of concrete
- Testing of pre-stressed anchors
- Testing of lifting hooks, anchors and embedded supports of equipment
- Functionality condition survey of some dam monitoring instruments.

The following civil structures of the HPP Piva were comprised by the surveying works:

1. Dam Body with galleries and shafts within it
2. Stilling Basin
3. Intake Structure with Valve Chambers
4. Headrace Tunnels
5. Powerhouse and Transformer Premises
6. Draft Tubes
7. Lower Surge Tank
8. Tailrace Tunnel
9. Grouting Galleries
10. Switchyard
11. Appurtenant tunnels and galleries.

A significant part of the works contracted for establishing the condition of civil structures were specific preparatory works, which should have enabled performing of the testing works. First of all, this refers to the works related to stilling basin dewatering and cleaning of the deposits. A special program was prepared for surveying of the stilling basin, which has never been dewatered during the HPP's operation period, as well as for establishing the condition of drainage system in its background. The fact that the user does not have "as-built" documentation available, additionally complicated implementation of the works, therefore the Contractor was not in a position to anticipate all possible problems that might occur.

Within the Tender Document, the quantities of the specified tests were tentative. The final quantities should have been evaluated on the basis of the completed works.

2.1.1 *Visual inspection*

In general, within the Tender for Civil Works all visible concrete and metal surfaces had to be inspected visually. The visual inspection of concrete structures should result in the detection of damage of the main load bearing elements of structures, such as: presence of cracks and deterioration, mechanical damages, erosion of surfaces, corrosion of steel, no cover on rebar or visual rebar, presence of moisture and leakages.

The purpose of visual inspection of metal structures is to reveal the location and nature of defects and deformation, determine shape, thickness and suitability of corrosion protection, conditions of sealing parts and tightness of bolted connections and to determine suspicious location which should be checked additionally.

All damages of concrete and metal structures, except for cosmetic, should have been noted on inspection drawings, to register those location and size. The photographs should have been taken.

The visual inspection of the headrace tunnels, draft tubes and stilling basin were to be performed only after those structures have been dewatered, whereas for lower part of the surge tank and tailrace tunnel, the visual inspections should have been carried out by divers. The video recording of all of the inspected areas by divers was requested by Tender. The powerhouse above the generator level and ceiling of switchyard cavern were out of the scope of the services due to existing new metal lining.

2.1.2 *Interior condition survey of concrete*

Interior condition survey of concrete includes the tests of physical-mechanical properties of concrete and environmental impact on it, more than 30 years after construction. Tests of concrete should have been performed by combining a non-destructive method (Schmidt Hammer generally) and a destructive method (sampling). Tests of environmental impact on concrete involve determining the concrete carbonization depth. The ultrasonic testing of some cracked zones of main importance has been comprised by the works.

Tender Document defined that the minimum number of core sampling tests should be 3 per structure. The samples were taken from the stilling basin, valve chambers, powerhouse, draft tubes, lower surge tank, switchyard and grouting galleries.

Having in mind that in 2007, within the Project—Study of Static and Dynamic Behavior of Mratinje Dam, 41 samples were taken from the dam body and tests of concrete were performed by using Schmidt hammer at around 400 locations on the dam, this type of testing has not been envisaged.

The Contractor was requested to make comparison between the obtained testing results and the testing results that have been carried out during the construction period.

2.1.3 *Testing of incorporated pre-stressed anchors*

Pre-stressed anchors, as a part of structure, were incorporated in the supports of the roof lining of powerhouse cavern and in the lower slab and walls of the stilling basin. The actual safety factors of these structures and definition of potentially required rehabilitation measures could be analyzed only based on the tests of present force in the anchors. The testing by non-destructive methods, were defined in the Tender Document.

It is important to point out that during the HPP's operation period up to now, it was very rarely necessary to activate the spillways, and in those situations the spillways mainly operated below the designed capacity. The dam middle outlets have not been opened so far.

2.1.4 *Testing of lifting hooks, anchors and embedded supports of equipment*

Testing of lifting hooks, anchors and embedded supports of equipment implies testing by test loading and ultrasound, radiographic testing of welds connecting the equipment girders. Identification of the element that should be tested should have been performed during the visual inspection.

2.1.5 *Functionality condition survey of dam monitoring instrumentation*

At the moment quite considerable part of instruments are out of order. This refers in the first place to the electrical instruments installed in the dam body. Also, measurements at certain number of instruments can be considered unreliable. Therefore, the Tender Document envisaged functionality condition survey of inverted plumblines, Rittmeyer installation and Interfels deflectometer. Clogging of holes of inverted plumblines and Rittmeyer water balance installation should have been surveyed using camera. The purpose of these testing works was to assess the possibility of their rehabilitation for reliable operation.

A basic system of strong motion accelerometers were placed along the central cantilever and special care was paid to monitoring of induced seismicity. The quite clear phenomena of induced seismicity were registered during reservoir impounding. At the moment the installed seismographs are well serviced and in good working condition.

2.1.6 *Engineering geological mapping* (*remapping*)

The updated mapping of the unlined tunnels and galleries should have been done according to ISRM standard. Rock mass classifications should have been done according to Z.T. Bieniawski and Barton. The purpose of these investigations was the assessment of possibly necessary additional measures for excavation additional protection.

2.2 *Testing of reservoir sedimentation and storage capacity*

No analyses related to actually useful volume and silting of reservoir have been carried out during the previous period. Therefore, relevant bathymetric surveys were performed, which provided reliable data for future water management and energy analyses.

2.3 *Testing of unstable areas and grout curtain conditions*

2.3.1 *Investigations of the unstable areas*

Storage reservoir of HPP "Piva" is formed in deep canyons of the rivers Piva and Komarnica. Their directions are mainly meridian, deeply cut dominantly into massive Triassic limestones and less frequently in other coarse-grained clastic sediments of lower Triassic and Cretaceous flysch. Valley sides of these two rivers are very steep to vertical, reaching the height of 300–1000 m. They transform into extensive gently undulated surfaces of the Piva mountains. These are strongly karstified limestone platforms with plenty of karst depressions: karstic poljes, valleys, sink holes, swallow holes, caves, pits, caverns, etc.

Geomorphological properties of terrain which result from action of various exogenous geomorphological processes appear as genetically diverse formations and activities, such as: fluvial, glacial, glaciofluvial, diluvial, proluvial and colluvial. Endogenous-neotectonic activity has also influenced development of canyons, geomorphological formation and shaping of terrain and occurrence intensity of exogenous processes on canyonslopes.

In these geomorphological and geological conditions, during the HPP's operation period, a number of phenomena of active instability has been registered (landslides, rockfalls and talus) and/or conditionally stable slopes or potential landslides and rockfalls.

During the Study preparation phase a terrain reconnaissance of the Piva HPP dam and reservoir area was performed, intended to register and analyze occurrences of instability on the slopes and inclinations in the dam area, downstream of the dam and in the reservoir area.

All relevant morphological, geological, hydrogeological and engineering-geological conditions for genesis and development of instabilities were analyzed and a degree of hazard and risk was assessed. Based on Study conclusions, a program of detailed investigations was defined, results of which should enable preparation of the design for necessary protection and rehabilitation measures at endangered areas.

Seven locations of unstable terrain (which were assessed to have an impact on safety of people, structures and/or property) were identified within the dam and reservoir area (see Figure 5).

Two locations (marked as Locations 1 and 2 on the Figure 5), with the zones potentially endangered by rockfalls, were identified in the dam area. Taking into account that these two locations are in the immediate vicinity of the dam, powerhouse, switchyard and access roads, they are assessed to have high hazard and risk degree (see Figure 6).

Five locations with active landslides (marked on the Figure 5. as Locations 3, 4, 5, 6 and 7) were identified in the immediate impact zone of the reservoir. From the aspect of hazard and/or risk degree, all landslides were classified as Class II and III of hazards and risks.

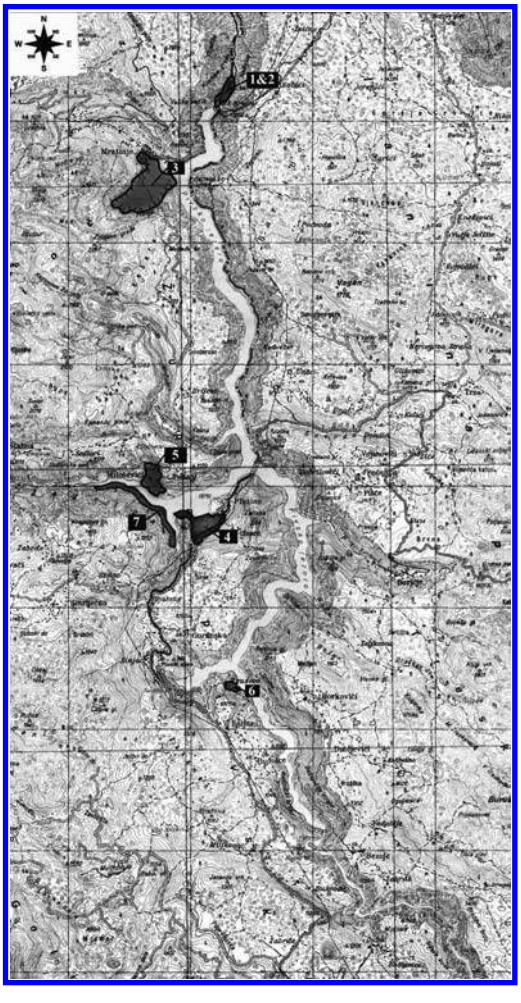

Figure 5. Unstable zones.

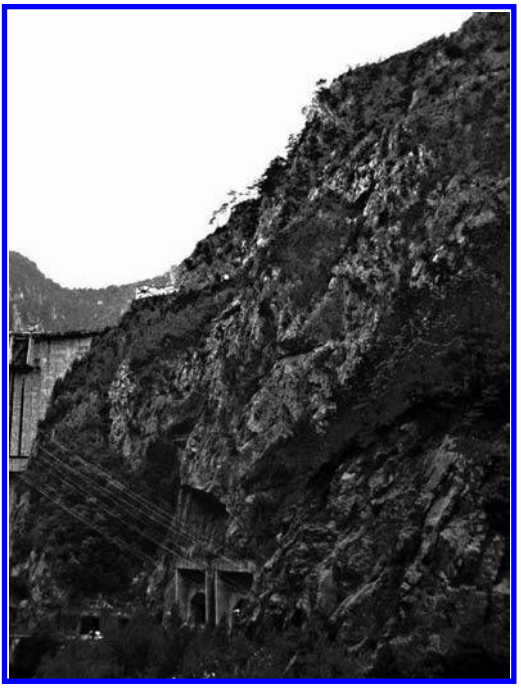

Figure 6. Slope above switchyard and entrance into underground powerhouse.

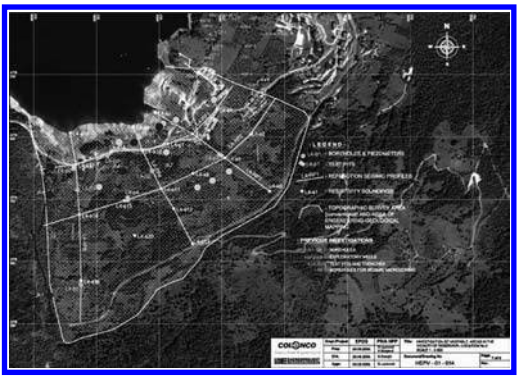

Figure 7. Location 4.

Based on the findings of geological and construction conditions and assessed hazard and risk degrees, adequate field investigations, including required auscultation measures, were defined for all identified instability locations.

2.3.2 *Measurements and investigations of grout curtain*

The grout curtain of the HPP Piva was designed and constructed in both abutments. The grout curtain in the left abutment is the three-row curtain

over the length of first 55 m and the single-row over the following length of 300 m. The grout curtain in right abutment is the three-row curtain over the length of first 95 m and the single-row over the following 360 m. The maximum depth of the grouting reaches 150 m below foundation level. The groundwater levels in the dam abutments are monitored in 56 piezometers (observation wells).

The following testing works were envisaged in the Tender Document:

- Measurements of the Piva River Discharges
- Analyses of Chemical and Physical Composition of Water
- Investigations by Applying Tracing Methods
- Investigations of Drainage System in Dam Abutments.

3 REVIEW OF CARRIED OUT TESTING WORKS

3.1 *Testing of civil structures*

Testing of the civil structures was performed in the period June–October 2010. Implementation of contracted works was conditioned by operation regime of the hydro power plant and that is why it was carried out in two phases. During the Phase I testing of structures which are accessible regardless of hydro power plant operation was performed, whereas the Phase II was implemented during the plant standstill period.

A detailed visual inspection of the structure (dam) of these dimensions required specific solutions. Inspection of the dam body from the downstream side was performed by the alpinists, who were directed by an expert team on the ground, by means of direct audio-visual connection. Inspection of the structures under the water was performed by the divers who were guided and directed in the same way. For special underwater inspections (at the depths exceeding 30 m), such as inspection of the upstream face of the dam and trashracks on the intake structure, the underwater robot was used.

Almost all the works required by the Tender Document were executed. Scope of the testing works, which was forecastly defined in the Tender Document, was adjusted to the condition of the structures that was found out at the site.

Based on executed visual inspections, classification of the noticed damages on the concrete structures was performed and adequate graphic documentation was prepared, in which locations of the damages were presented, with all their characteristics (type, dimensions, filling and moisture). In that way a kind of cadastre of the noticed occurrences was established. Unfortunately, after more than 30 years of the Piva HPP's operation, for some of the structures represents a zero state for monitoring the future behaviour of the civil structures.

At present, it is assessed that interventions in the dam body do not have to be envisaged within the rehabilitation works on HPP Piva. In the future, as in the previous practice, behavior of detected cracks should be observed during regular maintenance and if required the cracks should be repaired by executing adequate rehabilitation works. New leakages that might occur in the dam galleries and downstream dam face should be prevented by adequate grouting works executed within regular maintenance of civil structures.

The inclined cracks noticed in the dam abutments, in the zones of maximal stresses, have been separated as characteristic cracks. The ultrasonic testing of these cracks, performed by the alpinists, showed that their depth is up to 100 cm (see Figure 8).

A limited number of sampling and a lot of Schmidt's Hammer Tests were performed. Three (3) samples were taken from the upstream dam face. Strengths of concrete, obtained according to the EN 1992-1-1, are given in the Table 1.

The comparison between the obtained testing results calculated according to the JUS U.M1.048:1985 and the testing results that have been carried out during the construction period are given in the Table 2.

The following results were obtained from the samples that have been taken from the dam body in 2007 (according to the JUS U.M1.048:1985):

- Compressive strength is 45.51 MPa,
- Tensile strength by splitting is 2,56 MPa,
- Elasticity modulus of 33 years old concrete is 40.97 GPa.

It can be concluded that the performed tests confirmed the designed bearing capacity of the concrete civil structures.

Figure 8. Detected cracks in the right dam abutment.

Table 1. Summary of characteristic in-situ compressive strengths according to EN 1991-1-1 (MPa).

Structure	Designed compr. strength	Characteristic strength
Dam body—underwater CT	30	47.7
Stilling basin CT & SH	30	34.7
Valve chamber CT	22	33.4
Headrace tunnels SH	22	21.2
Powerhouse CT	22	27.5
Draft tubes SH	22	40.2
Lower surge tank CT		
– above water	22	45.0
– underwater	22	31.0
Tailrace tunnel CT	22	38.4
Grouting galleries CT	30	34.5
Switchyard CT	22	25.7
Appurtenant tunnels CT	22	25.5

CT—Core Test
CT & SH—Core Test & Schmidt Hammer Test
SH—Schmidt Hammer Test.

Table 2. Summary of characteristic in-situ compressive strengths according to JUS U.M1.048:1985 (MPa).

Structure	Estimated CS construction period	Characteristic strength
Dam body	42.9	65.6
Stilling basin	43.8	62.1
Powerhouse	31.5	46.0

Figure 9. Dewatering of the stilling basin and removal of deposited sediment.

As it was expected, a special problem during execution of the works was cleaning and dewatering of the stilling basin.

Material which remained in the stilling basin from the period of HPP's construction, mixed with pieces of rocks from dam abutments and the finest deposited sediment, made dewatering of the stilling basin by means of the pumps from the drainage pumping station, impossible.

With the assistance of divers the function of water pumping system was re-established, which enables dewatering of the stilling basin in future. In some way, this represents a rehabilitation measure. Although about 4,700 m^3 of deposits was removed from the stilling basin, it was not completely cleaned within the available time.

During visual inspections of the stilling basin, performed after its dewatering, no occurrences which would require rehabilitation measures were noticed. The exception is downstream cutoff wall, where breaking of the downstream concrete diaphragm occurred, way back in the period of HPP's construction. Remaining parts of this diaphragm were not removed before putting the plant into operation (see Figure 10).

Within testing of lifting hooks, anchors and embedded supports of the equipment, the following tests were performed: checking of hooks diameter; determining of carrying capacity of the

Figure 10. Broken diaphragm.

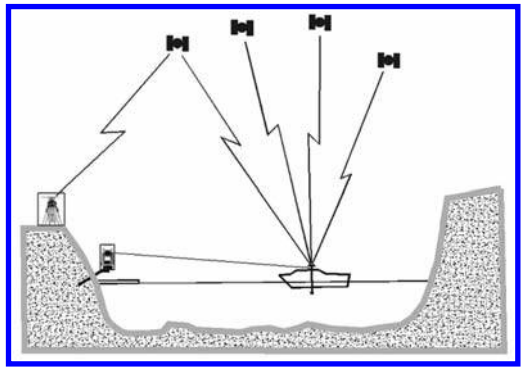

Figure 11. Position of the echo-sounder.

hooks by test load, thickness of metal sheets by ultrasound and load-bearing capacity of selected supports of the equipment in the switchyard.

Testing of the supports of equipment in the switchyard was particularly delicate task, with regard to the fact that testing was performed without removing of the equipment. It was concluded during successfully performed testing that the supports of the equipment have necessary load-bearing capacity, but they are affected to a varying degree by corrosion.

Due to small free length of anchors in the powerhouse cavern (control anchors were not provided) and embedded anchor heads in concrete lining of stilling basin, the testing of incorporated pre-stressed anchors could not be performed by a non-destructive method. That is why only two anchors—of the stilling basin and at portal of the switchyard (at which it was noticed during the visual inspection that their wires are outside the anchors) were tested by test load. The anchor in the wall of the stilling basin was not pulled out during its tensioning by the designed load, whereas the wires of the anchor at the portal of switchyard were broken.The condition survey of visible anchor head was performed as well.

It was concluded after performing the functionality condition survey of the dam monitoring instruments that the tested instruments are in a very bad condition and that they can be considered out of order.

After performing engineering-geological remapping, local zones were separated, at which it is necessary to apply additional measures for excavation protection, aimed in the first place to provide higher safety for the personnel during maintenance of the plant, but not to global stabilization of the executed excavations.

3.2 *Testing of reservoir sedimentation and storage capacity*

Bathymetric surveying of the Piva HPP's storage capacity was performed in absolute elevations and in the state coordinate network. Geodetic network consisted of basic and filling-in network. The basic geodetic network totally comprised 40 points, 17 trigonometric points, 2 reference points in WGS84 system and 16 newly established points. The accuracy amounted to 3 mm by position and 4 mm by height. Geodetic levelling comprised 29 bench marks at the stretch Mratinje—Pluzine, as well as technical levelling. The filling-in network consisted of 128 points with traverse lines: Mratinje—Pluzine, Vrbnica, Piva, Komarnica.

Surveying of the dam sections was performed by cross-sections, using the scattered points method, and along the characteristic terrain lines. The equipment consisted of: SM-26A specialized vessel, Trimble 5700 GPS and ATLAS DESO 300 echo-sounder. Echo-sounder's position was determined by GPS and total stations. Approximately 350,000 points were surveyed in total, i.e. 1 point per 1 m^2 of the terrain.

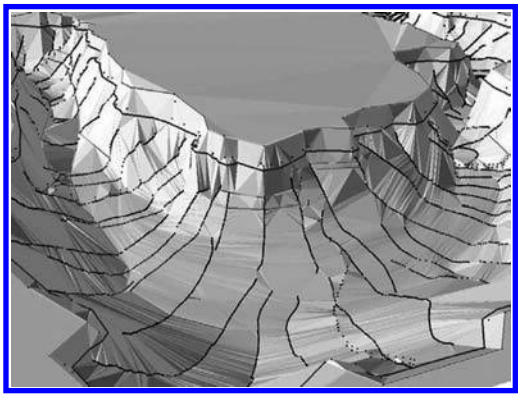

Figure 12. Preparation of Digital Terrain Model (DTM) for reservoir.

Preparation of Digital Terrain Model (DTM) of the reservoir bed was preceded by integration of the data collected by different geodetic methods, as well as filtering and adjusting of the data. Based on DTM, volume of the reservoir was calculated and cross-sections were prepared (Figure 12).

The storage capacity amounting to 824.557,000 m^3 was obtained for normal water level elevation of 675.25 m asl. The obtained storage capacity is less than so far valid capacity—amounting to 890 million cubic meters (determined at the time of preparation of the map in scale 1:25000), by about 55 million cubic meters (by about 6%).

The performed bathymetric surveying actually represents "zero" state of the Piva HPP's reservoir, with regard to the fact that this was the first, reference surveying. In future, it is necessary to monitor the reservoir condition periodically.

3.3 *Testing of unstable areas and grout curtain conditions*

The performed testing of unstable areas comprised the following works: topographic surveying of the terrain and scanning of steep slopes of dam abutments, engineering-geological mapping of the terrain, geophysical investigations, field drilling works, Standard Penetration Tests (SPT), pits excavation and sampling, as well as laboratory analyses and tests.

Types and scope of field and laboratory works were adjusted to morphological, geological, engineering-geological and hydrogeological properties of the terrain and they enabled obtaining of the data relevant for assessment of: stability and condition of the subject locations,

dam abutments stability condition and stability of the slopes downstream of the dam.

Investigations of grout curtain condition were performed in accordance with the Tender requirements.

Ground water tracing test was performed after performing the preparatory works for flushing of the piezometers which are a part of observation network. The testing procedure was performed in accordance with the instructions from the Tender Document.

According to testing results and so far performed analyses, the grout curtain is not completely watertight. Water communication has been established upstream and downstream of the grout curtain, in right and left dam abutments—partially through and around the grout curtain.

Tracing test results are very important and they represent a valid basis for making conclusions related to grout curtain condition, which will be elaborated in the following phase of Design Documents preparation, together with the proposals for grout curtain reparation, i.e. for improvement of its anti-filtration properties.

4 CONCLUSION

Results of condition surveying works of civil structures within the Piva HPP, confirm that condition of these structures is satisfactory and that load-bearing, structural elements are in relatively good condition. Registered occurrences of damages do not have an impact on stability and safety of the structures, but they have influence on their usability and lifetime.

In the future, as in the previous practice, behaviour of detected cracks should be observed during regular maintenance and if required the cracks should be repaired by executing adequate rehabilitation works. New leakages that might occur in the dam galleries and downstream dam face should be prevented by adequate grouting works executed within regular maintenance of civil structures.

The preliminary list of rehabilitation measures for arch dam includes the following:

- Rehabilitation and modernization of the dam monitoring system, including implementation of remote readings. The introducing of new generation of seismic monitoring equipment will be analyzed;
- Construction of protective structure, which will enable safe access for people and vehicles to the underground structures of the hydro power plant;
- Stabilization measures against movement of rock masses, for active landslides with III degree of risk (particularly at Pluzine settlement);
- Establishing of technical monitoring system at registered unstable areas;
- Rehabilitation measures of grouting and drainage curtain will be defined based on the results of the prepared related study taking into account the long-term behaviour of the piezometers and drainage water measurements and results of recommended tests.

Results of performed investigation works provide sufficient data for defining necessary rehabilitation measures. At the same time, these results represent starting point for future establishing of the Piva HPP's Maintenance Management System.

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Dam operation and maintenance in Iran

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ABSTRACT: Challenges in dam construction, exploitation, maintenance, inspection and repair activities as well as the experiences gained in these areas will be further discussed in detail (with reference to actual examples) in a paper following this abstract and certain suggestions will be proposed for holding the International Congress on Dam Maintenance and Rehabilitation.

1 INTRODUCTION

The amount of rainfall is limited in most parts of the world and this limitation is more visible in a country like Iran which is geographically located in an area with arid to semi-arid climate. Unbalanced rainfall, ranging from 30 mm in deserts and their outlaying regions to 1,800 mm in northern regions around the Caspian Sea, causes dual limitations with respect to availability of water. Dispersion of cities and villages, the need for water to raise agricultural products, and the need for drinking water have all given a special status to dams to play a role in satisfying the need for water and settlement of populations as well as to manage water resources.

Presently some 572 small and large dams with an approximate regulation capacity of 30 billion cubic meters are being exploited in Iran and a further 439 dams with a regulation capacity of 23 billion cubic meters are under study and construction. Since dams and their peripheral installations are very expensive and capital-consuming, it is of vital importance to preserve, maintain, rehabilitate and repair them in order to save the investments made on them and to guarantee public health and welfare and environmental freshness. However, proper maintenance and improvement of dams and their peripheral installations is faced with a number of problems and challenges including natural challenges such as damages incurred as a result of floods and inflow of sediments into dam reservoirs (as in the case of Sefidrood Dam in north part of Iran), great fluctuations in water flows as a result of harsh recurrent droughts, earthquakes, etc. There are still other challenges stemming from technical management and exploitation of dams during various stages of designing to construction and operation stages. Improper comprehensive and scientific inspection and maintenance plans, irrelevant annual budgeting and finance, recruitment of skilled technical supervisory and operational personnel. Lack of attention to measurement systems and telemetry, and lack of proper technology for carrying out inspection and maintenance activities are among other challenges in this area.

Regarding the fact that Iran has been among the 3 great dam-builders of the world, it has gained invaluable experience in construction and operation of various types of dams. Also, it has obtained valuable experience as an active member of ICOLD by exchanging expertise knowledge and experience with the other member of this Committee. Moreover, Iran has its own unique experiences for exploitation and maintenance stages of dam building.

Iran has had a significant breakthrough in the field of study, design, and implementation of large water supply structures, especially large reservoir dams following the Islamic Revolution in 1979. Such an achievement is due to the facts some of which are stated below:

1.1 *Climate condition*

Iran is a vast country with an area of 1,648 km^2 and varies in climate. However, it's mostly (about % 84) arid and semiarid. There are extensive precipitation variations during four seasons of a year. A part of Iran consists mostly of deserts and has a warm climate. Mean precipitation is 251 mm and the annual renewable water is about 130 billion cubic meters.

Spread of urban and rural areas, diversity of plants, cultivations, and bio-species and also, keeping a logical balance in environmental conservation, paying attention to the development of developmental infrastructures and social welfare are the reasons of dam construction in Iran. There are 572 dams with regulated capacity of about 30 billion cubic meters under operation and 439 dams with regulation capacity of 23 billion cubic meters of water under study or under construction (Figure 2).

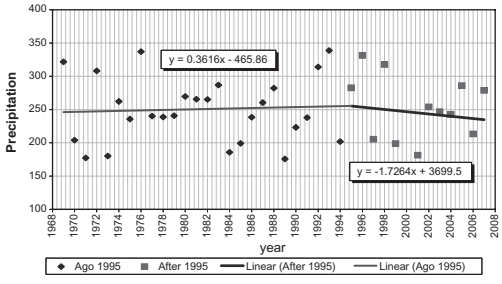

Figure 1. Trend of precipitation in Iran (1969–2008).

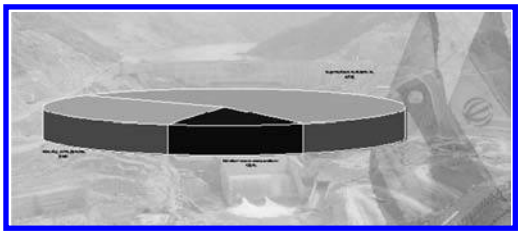

Figure 2. Operational, under construction and under study dams.

1.2 *Long background of water engineering measures and construction of major hydraulic structures in Iran*

In the last different eras, despite the limitation of facilities and technological capacity, hardworking people of Iran have been constantly attempting to overcome threats and water resources natural limitation and to use its interests. They have made special engineering designs and built valuable constructions by creating various innovations for meeting their needs. Dam construction, polband,[1] water network, water mills, water tanks, aqueducts, and Qanats are some of these constructions. Some noticeable instances of the remained constructions consist of 3500-year-old Chaghazanbil Water Treatment and Refining Structure, 2000-year-old Bahman Diversion Dam, 1700-year-old Mizan Diversion Dam and Shoushtar Water Mills, 1000-year-old Diversion Dam, 5000-year-old Zavareh Esfahan Qanats, 2500-year-old Gonabad Qanat, 700-year-old Korit Tabas Dam, 400-year-old Isfahan's Siosepol Polband and so on.

1.3 *Current needs*

precipitation variations, arid and semiarid climate, country's special geographical situation, high evaporation of water and intermittent drought from one side, and required drinking water supply which is dispersed in many rural and urban areas and also water supply for living needs such as agriculture, industry and environment brings about the necessity of dam construction for balance between water supply and its use. Therefore, dam construction and operation has a key role in the country's water resources management.

1.4 *Water engineering knowledge and raising dam construction capacity*

Although Iran has always been one of the water engineering activities' pillars because of the abovementioned conditions, it has used developed country's knowledge prior to the victory of the Islamic Revolution in the last century. Fortunately, from 1979 (Iranian year 1357), there have been great determination and engineering knowledge development, accepting the costs of investment and technical staff training, to reach self-sufficiency in complex hydraulic structures design and construction. Today, Iranian consultants and contractors have built numerous complex and large structures in this field. Besides, they have participation in other countries' work market.

2 ACCURATE AND OPTIMUM DAM OPERATION

Accurate operation and safety instructions and standards preparation are among the most important issues in safety and operation of dams. Dams are of extensive benefits and considered complex and costly structures. Inaccurate operation and occurrence of breaking or serious damages in dams are dangerous for dam's safety and environmental deterioration, entail large costs and cause public dissatisfaction. Dangers threatening heavy dam structures include inaccurate operation and dam breaking which could result in mortality. Operation and inspection of dams and their installations during operation period should be done regularly and annually to ensure dam strength and stability.

Major parameters affect the stability and durability of dams which include remedial work and timely operation and maintenance which is especially necessary for aged dams.

3 DAM MAINTENANCE SYSTEM

There are three kinds of dam maintenance which include routine maintenance, special maintenance, and strengthening maintenance. Routine maintenance consists of all the required works which can keep dam maintenance system in a desired condition and it's usually carried out annually. This system is implemented in a considerable number of dams. In special maintenance, natural and unpredicted

damage repairs such as damages caused by floods and earthquakes are carried out. This kind of maintenance is taken place in some dams. Strengthening maintenance is due to insufficient finance or facilities which lead to the postponement of maintenance measures over a year. This kind of maintenance is not an appropriate way and it's, in deed, an obligatory way because of lack of facilities.

4 CLASSIFICATION OF DAM DANGERS POTENTIAL IN IRAN

Dams are often in exposure of dangers with different degrees due to the potential of occurrence of dam breaking, flow of huge amount of water to reservoir and its spill, or inaccurate operation of dams.

In order to have more dominance on this issue, the way of classification of danger potential of dams has been explained below. For instance, classified system of danger potential of dams was defined by America's Federal Emergency Management Agency (FEMA) in 1994 and for further analysis; it has been classified into three levels of low, medium, and high danger potentials. In low danger potential, there aren't any losses of life, but few bioenvironmental and economical damages may be observed. Medium danger potential doesn't bring about any losses of life but it may cause damages and homelessness. High danger potential may cause both losses of life and bioenvironmental and economical damages. In our country, a considerable number of dams are nearly 40 years old, among them Golpayegan, Amirkabir, Dez, and Zayandehrood Dams can be named which need being defined and classified according to their danger potentials and remedial measures.

Fortunately, in recent years, effective steps have been taken on the field of operation and safety cares. Still, this classification has not been taken place with respect to the defined danger potential standards and there's a need to take a comprehensive measure. In this respect, preparation and collection of system's generalities "classification of danger potential for dams especially country's large dams, updated evaluation of dams' safety, compilation of major programs, and holding technical-applied workshops" are required.

5 STRUCTURAL AND NON-STRUCTURAL OBSTACLES IN DAM OPERATION BASIN

Basically, in operation and maintenance of heavy and costly structures of dams, recognition and attention to structural and non-structural measures and parameters is a necessary affair. In the section of structural measures, the main attention is focused on the physical measures such as the concepts of sediment, repairs, and so on. But in non-structural section, attention is mostly towards managerial and programming measures. For effective and optimum dam operation, both mentioned aspects should be taken into consideration.

6 STRUCTURAL OBSTACLES

There are a wide range of structural barriers and difficulties in effective dam operation and maintenance which are not the same in all dams. But rarely, a dam can be found which doesn't deal with any structural challenges. In the following sections of this paper, some issues are referred to which are related to the country's dam operation and their experiences can be useful for other dams. One of these barriers is sediment challenge.

7 DAM AND SEDIMENTATION CHALLENGE

Regarding to the arid and semiarid climate conditions in many regions of Iran, the pastures and forest cover are not suitable and a considerable amount of soil will be eroded with heavy rains and rainstorms and the eroded soil cause turbidity flows to dam reservoir which sediment in it.

The annual volume of sedimentation is 175 to 250 MCM. The rate of reduction of dam reservoirs capacity is annually %0.5 to %0.75 in Iran that will increase in future.

This phenomenon is similar to the loss of a large dam with 200 MCM of capacity in a year. The important point is that in addition to heavy costs of investment and loss of a part of dam objectives, new natural sites which are appropriate technical and economical places for dam construction decrease and the costs of selecting new sites strongly increase or sometimes it is not possible. In fact, a useless damaged dam or filled by sediments is not only a missed opportunity for downstream areas but also it's a threat for them.

Mechanical equipments and water hydrodynamic energy are used to remove sediment in dams in Iran. In the first method, efficiency is limited and low and its performance is long and requires the provision of machinery, removed sediment location, and acceptance of sediment transportation costs and etc. that it is not economical. In 1989, the experiences of Ekbatan dam extracting sediment in Hamadan province revealed that the first method is ineffective in extracting sediment from dam's lake.

In the second method which is more effective, we had many good experiences in Sefidrood Dam in Gilan province of Iran that will be explained in details in this paper. This method is also faced with some restrictions which their removal needs preparation, appropriate and practical planning and environmental justification.

One of the appropriate methods according to dam conditions is timely dense river inflow_evacuation through bottom outlets to downstream. This way can create appropriate speed in relative erosion of the part of upstream sediments and removed sediment. Of course removed sediment in dams confront with many side affects such as: conveyance of turbidity flows with high density to downstream zones and causing ecosystem deteriorations, damage to aquatic, accumulation of mud in the path of canals and inlet of irrigation networks, blocking the bottom outlets due to sliding of loose sediments from the layer of evacuation cone, erosion of sediment evacuation installations especially dam gates. Therefore one of the challenges and important problems in dam sustainability is sediment which is a major factor in deduction of the life of dams and a restriction to the achievement of the forecasting purposes. We have several valuable experiments related to problems and challenges to dam sediment in Iran for many years. One of the experiments is Sefidrood dam in North of Iran.

8 SEDIMENT CHALLENGE AND ITS SOLUTION IN SEFIDROOD DAM

Sefidrood dam is a sustainable gravity concrete dam which is constructed near Manjil city in Gilan province. The operation of this dam has started since 1942 (1341 Hegira) and its main purpose has been regulation of inflow for irrigating 150,000 Hectares areas in downstream and generation of 88 MW hydropower energy until 1980 (1359 Hegira). About 700 MCM of sediment accumulated in dam reservoir during 17 years and dam capacity deducted to 1150 MCM.We have annually evacuated the new interring sediment flow and about 20 MCM old sediment in order to solve this problem through engineering and performance measures from 1980 (1359 Hegira) to 1991 (1370 Hegira).

The total area of Sefidrood basin is about 5600 Km^2 and flowing annually about 4500 MCM volume of water into the dam. This basin has poor vegetation as a result of the rate soil degradation and transportation of transferred sediment is high. The annual rate of transportation of transferred sediment is about 43 Million Tons, therefore during 42 years the total inlet sediment and outlet sediment had been about 1850 and 1150 Million Tons respectively. Therefore, dam capacity in construction time has reached from 1760 MCM to 1150 MCM at present.

As mentioned above, Sefidrood dam is a gravity concrete dam with height of 6 m from foundation, crest length of 425 m, crest width of 8 m and total evacuation capacity of 6200 m^3/s (free morning glory weir and 14 evacuator canals). Thus the above situation and an experience which we have gained from of Reservoir Sediment flushing through bottom outlets were successful. However, costs and environmental supervision of this action is important. We can use above experience in similar situations. But the important and fundamental point is that sediment flushing is a remedial measure but preventive measures are very important which can be done through watershed measures.

9 THE IMPORTANCE OF MONITORING IN OPERATING OF DAMS

The principal and sustainable operation of dams will be determined in its design period and is operated and warranty by implementing telemetric and telecontrol facilities. So design and performance of monitoring structure and instruments of dams

Table 1. The main parameters for supervising the dam behavior and its foundations.

Concrete body	Earth body	Foundations
Structural deformations	Body deformations	Deformations, movement of abutments
Special movements (Joint & Crack)	Special replacement (concrete structures)	Special settlement (Cracks)
Temperature of dam	Body temperature for recognizing leakage	Body temperature for recognizing leakage
Uplifting pressures (concrete foundation & into the rocks)	Pore water pressure in body and piezometric balance	Dam body pore and uplifting pressures in the deep zone
The rate of leakage & drainage	The rate of leakage & drainage	The rate of leakage & drainage
Leakage chemical analysis and turbidity	Leakage chemical analysis and turbidity	Leakage chemical analysis and turbidity

especially using new technologies are as important as designing complex structures of dams and related structures is also important.

Attention to the selection of measuring instruments is of great importance and it must have necessary technical aspects and easy operation. Sensitiveness of devices for measuring and reading and their resistance against external factors are among the specifications of these instruments. The following table shows the main parameters for supervising the behavior of dams and their foundations.

Dams monitoring in Iran include two essential factors; one factor is division and installation of equipments which are performed according to engineering standards in most of dams and the second factor is operation of these equipments in which significant deficiencies are observed. These deficiencies are training necessity human resources, attention to time measurements and data analysis especially for ancient dams.

10 NON-STRUCTURAL OBSTACLES IN DAMS OPERATION

One of the non-structural restrictions is managerial view in paying attention to priorities of operation of existing installations relative to new installations, because by investment in construction and completion of a heavy structure of dam and related installations, it has been assumed that the main work is completed, while the main work is started from the initiation of operation. There are heavy financial investment, employment of a lot of engineers & technicians, employment of hundreds of labors along with plans and exact CPM (Critical Path Method) during the construction process. But financial resources challenges for operation and maintenance, problems of human resources and lack of enough managerial considerations will be revealed after dam completion and initiation of operation. Dam structures usually are constructed during 4 to 6 years period but should be operated about 50 year. Therefore, we require more attention in operation period. Following factors are emphasized in this respect:

- Sufficient prioritizing to operation by senior managers of companies
- Preparation of comprehensive operation plans
- Preparation of comprehensive software for operation especially in repairs
- Training and giving enough motivation in operators
- Preparation of applicable operation and maintenance guidance and performing them
- Timely inspection, supervision and repairs based on scientific and preventive methods.

11 CONCLUSION

Islamic Republic of Iran is active in dam construction which has constructed a lot of local large dams and generalized dam technologies into the country in addition to cooperation in dam construction and related installation in other countries. But we also require doing a lot of works in operation of dam. We have also acquired numerous experiences in sediment management, monitoring, dam operation and maintenance and safety methods. Still, a lot of works should be taken place in the field of economy of operation, participation of private sector and etc. Besides, paying attention to following factors is a part of pursuing and emphasized programs.

- Practical priority to operation in based on annual program and budget
- Classification of risk potential for dams especially large dams
- Promotion of client power and attracting of experts and incentive labors
- Producing suitable structure for operation, technical inspection and supervision system
- Reformation operation methods and processes and doing renewed engineering
- Utilization new technologies specially development of telemetric and telecommand
- Privatization dam construction industry
- Implementation of effective watershed plans.

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Rehabilitation of dams using in-house contractors: Lessons learnt from the contractor's perspective: A case study from South Africa

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ABSTRACT: An overview is given of current and recent rehabilitation projects of dams in South Africa, with regards to the division of work between external contractors and the in-house construction division. The rationale for giving some of the work to the in-house contractors is explained. A case is made for the existence of an in-house contractor to carry out at least some of the work. This is motivated by looking at a number of case studies where the in-house contractor was used. These case studies indicate that there are some instances where it is in the interest of the client to use in-house contractors. These include those projects where there is a great deal of uncertainty which could lead to delays, claims and dramatic changes of scope. It also includes instances where time is of the essence, such as where emergency repairs need to be done on dams. A case can also be made for using the in-house contractors where there is the absence of or limited supervision by a Resident Engineer. It is pointed out that the in-house contractor should also have other work, so as not to become a burden on the dam-rehabilitation division in times where there are not enough available projects. The need for a separate, flexible set of conditions of service is highlighted as one of the requirements for the success of such in-house contractors, along with a degree of separation between the contractor and client, even if both belong to the same organization. Some of the problems experienced are highlighted, and suggestions made to prevent similar occurrences.

1 INTRODUCTION

The Republic of South Africa is situated at the southern tip of the African continent. Although South Africa is by far the biggest economy in Africa, and ranked 25th in terms of GDP in the world in 2008, there are a number of challenges facing the country. Unemployment is high, and income inequality is approximately equal to that of Brazil.

1.1 *Construction in South Africa*

The state owned electricity supplier, ESKOM, started experiencing a lack of capacity during 2007. This entity then embarked on a massive construction program. In 2004 the hosting of the FIFA World Cup of Football (soccer), to be held in 2010, was awarded to South Africa. This meant that 5 new stadiums have been built, and another 5 upgraded. Public transport infrastructure was upgraded to accommodate the expected inflow of tourists, with projects such as the Gautrain and the Rapid Bus Transit system. The South African government, in an attempt to create employment opportunities and to stimulate economic growth, embarked on a large scale infrastructure investment program, which saw the construction of numerous new roads, airports and harbors, as well as extensions, upgrades and rehabilitation to existing infrastructure. This meant that the country experienced a "boom" in construction activities, after several decades of very low investment levels in infrastructure. This sudden acceleration in construction activities has led to severe shortages in skilled personnel and certain resources, such as cement, in the construction industry in South Africa.

1.2 *Dams in South Africa*

According to Sawadogo (2008: 3), South Africa has more than 500 government dams which have a total storage capacity of 37 000 million m^3. On average, this means South African dams have a combined ability to store approximately 74% of the Mean Annual Rainfall (MAR) of 50 150 million m^3/a. Bohensky et al. (2006: 28) described the historic development of storage dams and the future development of new dams in South African as follows: "Cumulative storage dam capacity in South Africa increased steadily from 1900 to 1975 and then increased sharply between 1975 and 1990, appropriating even a larger share of the total freshwater supply. Only in the 1990's did growth slow significantly

as a result of the saturation of available dam sites, along with the increasing acceptance by water managers that there was little water left to allocate, and that the actual cost-benefit ratios of large dams were rarely as low as originally projected".

The potential to construct storage dams with acceptable cost-benefit ratios in South Africa is declining mainly caused by the unavailability of suitable dam sites and the availability of surface water.

1.3 *The National Water Resources Infrastructure branch of the Department of Water Affairs*

The government Department of Water Affairs is the custodian of dams in South Africa. One of the branches of this Department is the National Water Resources Infrastructure branch (NWRI). The NWRI is made up of 4 Chief Directorates, namely the Chief Directorates (CD) Construction Management, Operations, Engineering Services and Project Implementation. While the CD: Construction Management (CD:CM—referred to as the in-house contractor in this paper) is tasked with the construction and maintenance work of water related structures, the CD: Operations operates the water related structures (mostly dams, canals and pump systems) and thus constitutes the operator and client when refurbishment is undertaken on dams.

1.4 *The Chief Directorate Construction Management (CD:CM)*

The in-house contractor is involved in a number of projects, for a number of clients, and across various disciplines. The in-house contractor employs approximately 4500 personnel, across numerous projects, ranging from small refurbishment projects, to large scale, multi-billion Rand construction projects where new dams and other infrastructure are constructed. Some of the current projects include the construction of the De Hoop Dam (an RCC dam which will provide water to platinum mines and households in the Mpumalanga and Limpopo provinces), the construction of the Nandoni dam and related purification and distribution works and numerous other construction and refurbishment projects. The in-house contractor works for various clients, both within the NWRI and in other spheres of government. Some of the clients include local government (municipalities) as well as other government departments, such as the Department of Agriculture, for whom irrigation canals are constructed.

1.5 *The Dam Safety Rehabilitation Program (DSRP)*

The Department of Water Affairs owns 350 dams nationally, 160 of which were identified as having safety-related shortcomings. The main reasons are that most of these dams were constructed before 1986, when new regulations and standards were issued, while limited maintenance has been carried out over the last 20–40 years. The problems identified are grouped into the following categories (some dams fall into more than one category, therefore the figures total to more than 100%)

- Insufficient spillway capacity 80%
- Slope stability problems 10%
- Inadequate maintenance eventually resulting in dam safety problems: 60%
- Geological/foundation problems: 10%
- Inadequate outlet systems: 80%

Source: (Muller: 2007)

It was realized that a major rehabilitation program would have to be embarked upon. Approval was received from National Treasury, and a sum of R 1 200 million (€ 120 million) was allocated for the initial stages of the Dam Safety Rehabilitation Program (DSRP). This program commenced in April 2005.

2 TYPICAL REHABILITATION WHERE IN-HOUSE CONTRACTORS HAVE BEEN USED

2.1 *Breakdown*

To date 34 projects have been completed under the DSRP. Of these, the in-house contractor was involved in 22, with the remaining 12 being carried out by private sector contractors. On some of the projects on which the in-house contractor was the civil contractor, private sector contractors were appointed to carry out some of the mechanical and electrical refurbishment.

2.2 *Specific projects*

When one drills down into the details of some of the projects which were undertaken, the reasons for having an in-house contractor become apparent. A summary of four projects is given below, with reasons why these specific projects lent themselves to being constructed by an in-house contractor. These examples are by no means exhaustive, but serve to highlight certain principles.

2.2.1 *Thabina—no full time engineer & emergency*

The Thabina Dam is situated in the Limpopo province, in the Thabina River, a tributary of the Letsitele River. It has a maximum capacity of 2.8 million m^3. The dam is 50 m high from river bed level to the Non-Overflow Crest (NOC) level

and it comprises a single curvature concrete arch with a gravity concrete spillway on the left flank. It was constructed in 1984.

In March 1997 serious flood damage was inflicted on the concrete apron and raw water pipeline downstream of the dam. After inspection of the flood damage it was evident that the measures against flood erosion needed to be upgraded. During the design of these works, the decision was taken to increase the capacity of the dam by raising the Full Supply Level (FSL), and to replace a pipeline which provides water to a nearby community.

The in-house contractor was tasked with the construction work. As Thabina Dam supplies water for domestic use to nearby villages, the paramount priority at the start of the project was provision of water from the dam to the purification works. A 450 mm steel emergency pipeline was therefore installed in the river bed over a length of approximately 300 m, replacing the damaged pipeline. As an alternative to a conventional stilling basin, a stepped spillway with a shortened basin and end sill was constructed to deal with energy dissipation.

It was realised that another problem, namely availability of water, could be overcome as part of the project. The local domestic requirements from Thabina Dam were estimated at 1.39 million m^3/a in 1995 (Beukes, 2005), with the total overall demand (irrigation requirements included) being 3.28 million m^3/a. It was expected that the requirements for domestic use alone would exceed the yield of the dam (2.92 million m^3) by 2015, after which the supply would have to be augmented from other sources. By 1999 the average discharge from the dam was already of the order of 10 000 m^3/d, which exceeded the yield of the dam. DWA's regional office therefore requested that additional storage be provided as part of the remedial works, if possible. It was determined that raising the FSL of the dam by 2.4 m would increase the capacity of the dam from 2.8 to 3.4 million m^3. (The height of the raising was restricted by flood discharge requirements). The inflow into the Thabina Dam was calculated as 5.54 million m^3/a (Beukes, 2005), which meant that after raising, the capacity of the dam would amount to 62% of the estimated annual inflow.

As a construction team with all the necessary equipment would already be established on site, the cost of raising the dam would be greatly reduced compared to this being done at a later stage.

The project was completed in April 2000. The work was carried out without a full time engineer on site, due to the fact that the in-house contractor had a competent and trusted site agent on site. Because the in-house contractor was utilised to carry out the work, it was possible to respond quickly to repair the flood damage, including the reconnection of the pipeline which supplies water to the surrounding villages.

2.2.2 *Thapane dam—emergency work*

The Thapane dam is located in the Limpopo province, on the Nwanedzi River. The dam provides water to a number of rural villages via a purification works and pipe network. Although the dam is relatively small, it is vital to the villages which depend on it for drinking water. In 2000, the eastern and northern parts of South Africa were hit by tropical storms, which caused widespread flooding. During these storms, the dam was seriously damaged, with the earth embankment losing most of its topsoil dressing, and major erosion taking place. Engineers from the Dam Safety office of the Department assessed the situation and it was decided that temporary repairs, to ensure that the dam wall did not break, were urgent. The in-house contractor was asked to carry out these repairs, which was done. Two weeks after the initial damage, 500 mm of soil was placed on top of the crest, which prevented the wall from being washed away during a second round of flooding in 2000.

Subsequently, and while the contractor was on site, it was decided to upgrade the dam, so as to ensure that it had a larger spillway, and to do some much needed maintenance and repairs. In the process the non-overspill crest was raised, thereby increasing the freeboard, the length of the overspill crest was increased, the spillway channel was widened, downstream filters, slope protection and seepage collection channels were installed, and a road bridge was built across the spillway. Previously vehicles and pedestrians crossed the spillway channel by driving through it.

The work done at Thapane dam is a good example of how an in-house contractor can be used to cater for emergencies, where there is often not time to follow normal procurement methods to appoint a contractor, and where no designs exist at the time of commencement of work. In this case, the designs were done on an ongoing basis, and were communicated between the site agent and engineer. Because the contractor's main priority was not to make profit, there were no claims for extensions of time. Because the contractor had other projects in the area, personnel and other resources could simply be moved to other sites when there was a delay in the decision making process as to how to proceed with a certain activity, to allow designs to be finalised. There was also no need to appoint a full time supervising engineer to represent the client or engineer. Because the work was done on an actual cost basis, this meant that the final cost of the project was kept to a minimum.

2.2.3 *Kammanassie Dam—ongoing work/ uncertainty*

Kammanassie Dam is a dam on the Kammanassie River, near De Rust, Western Cape, South Africa. It was constructed in 1923. The rehabilitation and maintenance work done on the Kammanasie Dam does not form part of the DSRP project, but is instead done as part of the normal maintenance schedule of the dam. The budget allocated to the Directorate: Southern Operations, which operates the dam, has to be divided between the maintenance of several dams, and as a result the maintenance of Kammanasie dam, although major in scope, has to be spread over a number of financial years. There is also the possibility that, due to changes in budget and priorities, the refurbishment might be stopped temporarily at short notice. This means that any contractor appointed to do the work, should be able to accelerate or decelerate at short notice, and perhaps even stop work. This should be done while keeping costs, including claims, as low as possible.

The in-house contractor has been working on the project since June 2007, during which time a number of phases were successfully completed. The individual phases have been implemented depending on the funds available in the various financial years. Some of the work done includes repairs to causeways, flank walls, the spillway crest and an existing canal, as well as the construction of additional downstream concrete protection slabs, an access road, gabion slope protection and the installation of a gravity pipeline.

Only actual costs have been claimed for work done, which meant that, despite the stop-start approach to the project, and uncertainties regarding funding, the refurbishment of the project could be completed within budget, and with no disputes regarding costs.

2.2.4 *Pongolapoort dam- mixture of certainty and uncertainty*

Pongolapoort dam is situated in the Pongola River in the Kwazulu-Natal province of South Africa. The dam was constructed in 1973. It is the 4th largest dam in the country. The capacity of the dam is 2 492 million m^3, with a maximum wall height of 89 m and a crest length of 515 m. It is a medium thin, double curvature arch dam with a gradual transition towards a gravity thrust block on the left flank

Phase 1 of the rehabilitation work encompassed the rehabilitation of the road bridge which crosses the dam wall. An external Professional Service Provider (PSP) was appointed by the Department to oversee the design and site supervision of the rehabilitation work. The work was outsourced to a private contractor, and this phase was successfully completed. It should be noted that the work was of a straightforward nature, and that it was possible to determine the detail of work to be done before the work commenced. It was therefore relatively easy to draw up a detail design and specification, as well as a Bill of Quantities. This made the tender procedure relatively easy.

Phase 2 of the rehabilitation work will not be so clear cut. This will entail the improvement of the reliability of the radial gates, as well as the alteration of existing access to the mechanical equipment for maintenance purposes. Because the extent of the alterations (and the precise state of the existing gates and mechanical equipment) cannot be accurately determined while the dam is in full operation, and in order to minimize downtime, the work will probably be given to the in-house contractor, who has a mechanical repair and refurbishment division. In this way, the design engineers (in-house) and contractors (also in-house) can liaise on site to find the optimum solution.

Phase 3 of the refurbishment will be a long term solution, where the hydrology of the dam will be investigated to determine if the spillway capacity complies with new safety regulations and standards. Because not all the original design criteria and calculations are available, the hydrology will have to be revisited. Should the need for modifications to the spillway become apparent, the manner in which this will be constructed will be informed by the availability of contractors, as well as by the availability of information beforehand, and other constraints, such as the availability of funds and other resources. There is a possibility that this work could be awarded to the in-house contractor, should the circumstances necessitate this.

3 LESSONS LEARNT FROM IN-HOUSE CONSTRUCTION

3.1 *Involvement in a number of projects*

As mentioned above, the in-house contractor forms part of the NWRI. The NWRI is a Trading Entity within a government department, which means that it should work on a cost recovery basis, and thus recover costs incurred through water charges. Because South Africa is a developing nation, the NWRI is faced with a number of customers who are resistant to increases in water tariffs. Therefore it is important to keep costs of rehabilitation and other projects as low as possible, so as not to escalate the eventual tariffs which customers pay for water. One of the ways in which the in-house contractor attempts to do this is to distribute overhead costs, such as costs relating to a Head Office component, over as wide a base as possible.

This means that the various units of the in-house contractor aim to be involved in various projects for various customers at any given time. By doing this, the in-house contractor is not only in a position to minimize the fixed overhead cost allocated to individual projects, but it is also able to better level resources and achieve a steady base of projects over a longer period. It has also been possible to achieve continuity and to retain skills. Working for various clients, allows the in-house contractor to widen its scope, attract and retain specialists over a wider field of expertise, and leverage economies of scale with regards to supply chain practices.

Currently the in-house contractor works for other divisions of the Department of Water Affairs, both within and outside the NWRI, for other government departments, for local government (municipalities) and for other organs of state. An added benefit of having a large base of clients is that the fixed costs of operating a construction division do not have to be carried by one division (such as a dam safety or asset management division) during periods of lower activity.

3.2 *Separate, flexible conditions of service*

In South Africa (as in most countries), being employed by government is seen as being a long term prospect. Most government departments in South Africa are responsible for the formulation and implementation of policy governing specific sectors. Government (both local and central) is one of the main clients, appointing mostly private sector contractors to carry out construction, maintenance and refurbishment work. In South Africa the Department of Public Service and Administration (DPSA) determines the conditions of service of most government employees. Most government employees are appointed on long term, pensionable contracts. Because of the project-based nature of the work carried out by the in-house contractor, the need exists to have a separate set of conditions of service for personnel appointed to carry out these projects. In the National Water Act of 1998, under article 76, provision is made for the creation of such conditions of service. Thus personnel appointed to carry out work for the in-house contractor are appointed on short term contracts, in line with the nature of the work. It is also possible to provide certain benefits to employees of the in-house contractor which would not normally be provided to other government employees, such as housing, transport and various allowances. The provisions of this separate set of conditions of service should also allow for different working hours than the rest of the civil service, easier transfers of personnel, as well as shorter appointment and termination procedures.

Without this separate set of working conditions, it would be very difficult for any government department to maintain a construction division. Succession planning based on a proper skills development program directly focusing on the specific type of construction work to be performed, is seen as a primary cornerstone in building and maintaining a capacitated workforce. The cumbersome, bureaucratic procedures normally followed in the public service, along with the rigid conditions of service do not lend themselves towards use in the construction environment.

3.3 *Delegation of authority*

One of the major problems experienced by the in-house contractor is that it operates as a contractor within a public service entity, and that this is not always appreciated. While the public sector is responsible for a large number of construction projects, the actual involvement of the public sector, as far as procurement is concerned, is normally limited to the appointment of a small number of PSPs and contractors via a tender process. A project valued at hundreds of millions of Euros might be procured utilizing as few as four or five tenders.

In contrast with this, the in-house contractor is responsible for a significant number of transactions per day. Everything from consumables through to construction materials and services have to be procured. In doing this, officials have to ensure that they remain within the stipulations of the procurement delegations of the Department, and of the government of South Africa. Projects, and especially projects involving Construction, are by nature prone to many changes. This is partially due to lack of complete information at the beginning of a project. Contractors can normally deal reasonably well with sudden changes, since all major construction contracts, such as FIDIC and NEC3 deal with issues such as payment for extra work and extension of time.

If provision is not made to accommodate unplanned changes to work, which necessitate procurement outside the normal procurement provisions of government, an in-house contractor will be inefficient as part of a public entity. One way to solve this problem is to delegate the authority to approve procurement, outside normal procedures, to individuals involved in the construction process. Typically a limit should be placed on the amount that a Contracts Manager, Director or Chief Director can approve. In cases where the amount exceeds that which can be approved by one individual, provision should be made for an extraordinary panel, which could approve procurement outside the normal procurement procedures.

It is important that both the panel and delegated individuals should be aware of the responsibility which accompanies their appointments, not only to ensure that the laws of the land are followed, but also as far as ensuring that procurement does not become a stumbling block in the efficient operation of an in-house contractor.

3.4 *Separation from client*

Although there are varying levels of formality when the in-house contractor is utilised by the various role players as a contractor, there is definitely a need to have some level of separation between the client and the contractor. Often, when the Department of Water Affairs is the client and contractor, the engineer, client and contractor work together very closely. Because the in-house contractor is not primarily concerned with profits, but more with quality and ensuring a long term client base, the need to take "short cuts" is not as pressing as might be the case with private contractors. The client and engineer thus trust the contractor to follow plans and specifications more readily than might be the case if a private contractor was involved. The existence of a well-define quality assurance plan and the effective execution thereof, is paramount in building and maintaining this relationship of trust. While there are several advantages to this approach, care should be taken to keep some level of separation between the various parties so as to ensure that responsibilities do not clash and that accountability remains with the respective delegated individuals. Although communication might flow very freely between the various role players, it is important to keep certain communications formal, so as to have a record of requests and decisions. It is important to keep accurate, detailed cost reports in order to be able to compare the performance of the construction unit with benchmarks and to be able to improve continuously.

3.5 *Importance of clear understanding of roles when third party engineer is involved*

While the importance of a certain degree of separation between the role players in the construction process is highlighted above, it is inevitable (and perhaps desirable) that, due to the fact that the contractor and client work for the same Department, some degree of relaxation of formal communication and reporting protocols will ensue. Because the same contractor and client work together over a number of years and across various projects, a sense of understanding and trust is bound to occur. When a new third party engineering company is appointed to supervise and design a certain project, it is important that this engineer understands the relationship between all role players. Often the engineer is unaware of the subtle deviations from the strict client-engineer-contractor relationship as is normally practiced. In order to avoid misunderstandings and the engineer being side-lined, it is important for the client to make sure that the engineer is aware of and understands the relationship before the project commences.

Another problem which became apparent during the DSRP was the conflict which exists in South African Dam Safety legislation, which calls for an Approved Professional Person (APP) to be appointed on most dam projects. Normally this person would be employed by the Department, and thus report to the client, and would not be part of the engineering team. This individual, according to legislation, takes full responsibility for the design and construction of the dam. Clearly this is not possible where there are different contractors and engineers involved. This also clashes with some of the stipulations of the FIDIC contract conditions which are normally utilised on these projects.

4 CONCLUDING REMARKS

There is definitely a role for an in-house contractor in a government Department, especially under specific circumstances. In a country such as South Africa, with skills shortages and limited resources, having an in-house contractor can enable a government Department to better plan and budget for critical rehabilitation, maintenance and construction activities. During periods of high activity in the rest of the construction sector, when contractors are prone to inflating prices due to full order books, a government department could ensure significant savings by making use of its in-house contractor. When limited or no information is available regarding old structures or when uncertain conditions prevail, it might be wise to make use of an in-house contractor to carry out the work, as this would almost certainly prevent expensive claims. In emergencies, it is also handy to have an in-house contractor who can be deployed at short notice.

The in-house contractor should however not be seen as a solution under all circumstances, but rather as a contractor to be used in specific conditions. Under conditions where there is adequate capacity in the market, and where most or all unknowns can be clarified prior to the start of construction, work can be outsourced to private sector contractors. Some mechanism should exist to ensure that the efficiency and rates of the in-house contractor are measured against those of private contractors, but at the same time the in-house

contractor should not necessarily compete with the private sector, since the justification for having an in-house contractor is not to simply create another contractor. Rather this contractor should be utilized in specific cases.

The Construction division within the Department of Water Affairs has enabled the Department to develop dam construction expertise which it has been able to retain in South Africa, even through cycles of loss of personnel due to emigration and other skills crises. The Department has also been able to develop capacity amongst engineers, technicians and artisans, much of which has filtered into the private sector.

It is worth considering the option of having an in-house contractor, but if the right circumstances are not created for this unit to function in, success is unlikely.

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Rehabilitación de la presa de Alloz (Navarra)

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ABSTRACT: Alloz dam, its reservoir and foot reservoir are located in the Foral Community of Navarra. Inagurated in 1930, it regulates the headwaters of the river Salado, providing resources for irrigation in the area and for the production of electricity. It is one of the first double-curved arch dams built in Spain. Due both to the need for adaptation of its elements to the new standards as well as to solve several problems arising from their exploitation, it has been necessary to undertake several remedial measures which are described in this paper (operation of bottom drains, fitting of the restitution channel and recovery of the river reach downstream from the foot reservoir and repair of irrigation intake after the avenue of 2007).

RESUMEN: La presa de Alloz, su embalse y contraembalse están situados en la Comunidad Foral de Navarra. Inaugurada en 1930, regula la cabecera del río Salado, aportando recursos para el riego de la zona y para la producción de energía eléctrica. Es una de las primeras presas bóveda de doble curvatura realizadas en España. Debido al tiempo transcurrido desde su construcción, a la necesidad de adecuación de sus elementos a la nueva normativa, así como a la solución a diferentes problemas derivados de su explotación, ha sido necesario acometer diferentes obras que son las que aquí se describen (operatividad de los desagües de fondo, acondicionamiento del canal de restitución y recuperación del primer tramo de río aguas abajo del contraembalse y reparación de la toma de riego tras la avenida de 2007).

1 LA PRESA DE ALLOZ

La presa y el embalse de Alloz están situados al Suroeste de Pamplona, Comunidad Foral de Navarra, en los términos municipales de Facero, Guesalaz y Valle de Yerri. La población más cercana a la presa es Alloz, de la que hereda su nombre.

La presa de Alloz regula la cabecera del río Salado, aportando recursos hídricos a las áreas regables dominadas por el Canal de Alloz y a la central hidroeléctrica a pie de presa. Posee un contraembalse desde el cual, y a través del canal de Alloz, se alimenta a la central hidroeléctrica de Mañeru.

El contraembalse se halla situado íntegramente en el concejo de Alloz, perteneciente al término municipal del Valle de Yerri.

Tanto la presa como el embalse y contraembalse se ubican sobre el río Salado, afluente del río Arga, que es a su vez afluente de Aragón. Al río Salado se le incorpora, el río Ubagua en la cola del embalse, siendo este último el principal curso de agua, en cuanto a la aportación de recursos hídricos al embalse.

Figura 1. Vista general del contraembalse.

1.1 *Antecedentes generales*

Los estudios para el aprovechamiento hidroeléctrico en el río Salado comenzaron hacia el año 1910 con la constitución de la Sociedad Hidroeléctrica de Alloz, la cual obtuvo la concesión de la explotación para derivar hasta 4000 l/s, presentando en

Figura 2. Presa de Alloz.

1912 un proyecto de aprovechamiento mediante la construcción de un simple azud de 2.20 m de altura sobre el nivel de estiaje, a 400 m aguas arriba de la cerrada que el cauce presentaba a 3.5 Km del puente de la carretera de Pamplona a Estella.

Tras varias modificaciones y propuesta de diferentes soluciones, en diciembre de 1929 se redactó el proyecto definitivo del embalse en que se contempla una presa bóveda de 60 m de altura.

Cuando se redactó el proyecto anterior, ya se había construido parte de una primitiva presa de gravedad (contemplada en una de las modificaciones intermedias), utilizándose el desagüe de fondo (túnel de 85 m de longitud y 1.20 × 2.00 m de sección) realizado en el estribo derecho a 0.50 m sobre el nivel de estiaje, para el desvío del río.

Finalmente, la presa de Alloz fue inaugurada en 1930, siendo una de las primeras bóvedas de doble curvatura realizadas en España.

1.2 *La construcción de la presa*

La construcción de la presa de Alloz se llevó a cabo por tongadas horizontales de hormigón, de un metro de altura, para conseguir un mejor control de ejecución y optimizar las características del encofrado.

El paramento de aguas arriba se revistió de una capa de Inertol (compuesto sellante formado por polímeros en dispersión y aditivos especiales).

Se construyeron dos juntas de contracción situadas de tal manera que sólo soporten esfuerzos de compresión. Estas juntas se pueden asimilar a dovelas de arco, con lo que al llenarse la presa, tienden a cerrarse y asegurar la impermeabilidad, además, en la parte de aguas arriba, se colocó cartón embreado a ambos lados de cada junta. Dichas juntas están centradas en el cuerpo de presa y separadas 27 m entre sí.

Las obras de construcción del aliviadero (ubicado en el crestón calizo del estribo derecho) se iniciaron a la vez que la ejecución de la presa, aunque, debido a variaciones en el proyecto y a los resultados de los ensayos hidráulicos en modelo reducido, cuando se realizó el primer llenado del embalse, todavía no estaban acabadas. Además, las obras debieron ser paralizadas en repetidas ocasiones por los repetidos vertidos que se produjeron y que ocasionaron importantes daños en el canal de descarga (cuya excavación se había iniciado). Todo ello obligó a reformar el aliviadero, modificar la galería forzada, el canal de descarga y el canal a cielo abierto.

1.3 *Características de la presa y del embalse*

La presa de Alloz es una de las primeras bóvedas de doble curvatura construidas en España. La altura máxima sobre cimientos es de 66.80 m y la longitud de coronación es de 74.40 m.

El espesor mínimo en coronación es de 1.50 m, aunque la plataforma superior para el tránsito peatonal tiene 1.90 m de anchura incluyendo la barandillas metálicas. El espesor máximo en la base es de 17,50 m.

Los paramentos son curvos, tienen distintos radios y una curvatura variable.

El máximo volumen de embalse es de 65.39 Hm^3 considerando el nivel máximo normal (468.69 msnm), y de 70.15 Hm^3 hasta la coronación.

Si bien no hay actualmente ninguna galería en la presa, inicialmente hubo una que fue taponada por pensarse que representaba un punto de atracción para las filtraciones.

1.4 *Tomas y elementos de desagüe*

El aliviadero se encuentra ubicado en el estribo derecho a unos 80 m del cuerpo de presa. Su posición es oblicua a la corriente y está controlado por tres compuertas iguales tipo *Stoney*, con tres vanos de 5.60 × 3.00 m dispuestos en forma poligonal.

Las pilas entre vanos están construidas con hormigón armado, tienen una altura de 6.60 m sobre el umbral del labio y se han revestido con una chapa cilíndrica de acero inoxidable para aumentar la capacidad de descarga del conjunto.

Los caudales vertidos por los tres vanos, convergen en una cámara de carga, a partir de la cual comienza el canal de descarga del aliviadero. La solera de la cámara de carga se encuentra 12 m más baja que la cota del umbral de las compuertas.

El canal de descarga está formado, en su tramo inicial, por una galería con bóveda semicircular de dimensiones 4.45 × 4.45 m. Sigue después con un tramo a cielo abierto y, finaliza con otra galería, similar a la primera, que desemboca en el cuenco de amortiguación.

El desagüe de fondo de la presa de Alloz se halla situado en la parte inferior central del cuerpo de presa, unos 5 m por encima del antiguo cauce.

Para permitir el desagüe de esta instalación y el acceso al conjunto, se excavó en el cuerpo de la antigua presa de gravedad una galería de 1.60 × 1.20 m y con bóveda de medio punto. Actualmente, esta galería es visitable y permite el acceso al paramento de aguas a bajo de la presa, en la salida de la tubería de desagüe.

Tras varios años fuera de servicio por una obstrucción, se realizó un importante acondicionamiento del desagüe de fondo que aprovecha únicamente la tubería embebida en el cuerpo de la presa (de 1.5 m de diámetro y 17.5 m de longitud) a la que se añade un conducto de diámetro 1000 mm inmediatamente agua abajo del paramento de la presa, donde se forma un codo para bajar la cota del eje 2 m. La válvula de cierre es de tipo Bureau (de dimensiones 1.25 × 1.50 m y está dotada con sus correspondientes dispositivos de cierre y aireación, mediante una válvula ventosa de 300 mm de diámetro y una válvula de regulación tipo Howel-Bunger. Todo el conjunto de válvulas se acciona mediante un dispositivo electro-hidráulico y se alojan en el interior de una caseta de dimensiones en planta 9.50 × 6.00 m y una altura de 8.75 m.

Las tomas de agua existentes en la presa son dos, una destinada a los riegos y otra de alimentación de la central hidroeléctrica.

La toma de riegos está situada debajo del aliviadero y la forman tres conductos metálicos de 1 m de diámetro (cada uno de los cuales está cerrado por una válvula de seguridad y una de maniobra) que descargan al cuenco del aliviadero.

La toma para la central está protegida por una reja metálica y se encuentra situada en la margen derecha a unos 10 m por encima de la cota de cimentación. Tiene una anchura libre de 3.50 m con un perfil abocinado de 12 m de altura libre que se va reduciendo hasta alcanzar una sección circular de 3,50 m de diámetro al comienzo de la galería forzada.

La solera de la toma, hasta este punto en que comienza la galería forzada tiene una longitud de 24 m. Al no existir ataguía de cierre, toda la toma y la galería forzada están siempre en carga.

Tras la galería existe una tubería forzada de 2.30 m de diámetro y una longitud de 67 m, antes de alcanzar el edificio de la central.

1.5 *Central hidroeléctrica*

La central hidroeléctrica de pie de presa es propiedad de Iberdrola, encontrándose en explotación desde el año 1930. Junto a ella se encuentra ubicada una subcentral de transformación.

Se trata de una central exterior dotada de dos grupos hidráulicos con una potencia instalada de 8400 KVA y una producción media anual de 16,340 MWh.

Cada grupo está formado por una turbina de eje horizontal tipo Francis, y un alternador de la marca Oerlikon Suisse. La potencia de cada turbina es de 3850 CV, y el alternador es de 3000 KVA.

1.6 *Incidentes históricos*

Desde la terminación de la presa en 1.930, ya se observaron filtraciones que aumentaban con el tiempo y con el llenado del embalse, provocando una primera campaña de inyecciones, las cuales se sucederían a lo largo de los años hasta 1.970 en que culminó la última.

Las filtraciones, acompañadas en ocasiones por daños en la pantalla de arcillas, se producen esencialmente por disolución de yesos, siendo éste el principal problema de la presa de Alloz.

Como consecuencia de ello, la galería utilizada para el desvío del río durante la construcción de la presa, fue taponada al pensar que representaba una línea de atracción y facilitaba el aumento de filtraciones en la zona de yesos. Inicialmente se taponó aguas arriba pero al seguir existiendo elevadas filtraciones se dispuso, aguas abajo, una válvula compuerta. Tampoco resultó eficaz este sistema y en los años 90 se eliminó dicha válvula y se acondicionó el tapón de cierre.

Para evitar el contacto del agua con los yesos del cimiento del estribo derecho, se protegió este con una pantalla de muros de hormigón separados 6 m y con una altura de 3.5 m entre los que se extendieron capas de arcillas de 1 m de espesor, recubiertas por escollera de protección.

La boca del desagüe de fondo se encuentra encajonada con la ataguía construida para el desvío del río y que está muy próxima al paramento de la presa.

La instalación inicial del desagüe de fondo estaba proyectada para descargar directamente al cauce. La conducción del desagüe era de 1 m de diámetro pero estaba cerrada por una válvula compuerta tipo mariposa de diámetro 1.73 m, que había sido comprada con anterioridad a la construcción de la presa. Ello obligó a la colocación de una transición entre conducto y válvula, mediante un tubo cónico y otro cilíndrico.

Funcionalmente, el vaciado del embalse necesitaba de la operación de la válvula, de forma que el agua discurría por la galería de la antigua presa de gravedad hasta el cauce del río.

En su día, con motivo de una revisión que debía realizarse en la válvula del desagüe de fondo se intentó conformar una ataguía mediante el empleo de una bola de acero. Ésta estaba sujeta por dos cables a dos puntos fijos de los estribos, aguas arriba. El cable de la margen izquierda se rompió y la bola quedó encajada en el conducto, sin posibilidad de movimiento, con lo que el desagüe de fondo

quedó fuera de servicio. Esta era su situación hasta que se procedió al vaciado del embalse para la puesta a punto del desagüe de fondo en 1995.

2 LA SITUACIÓN EN EL CAMBIO DE NORMATIVA

Como consecuencia de lo expuesto, en el entorno del año 1.995, en el que se renueva en España la normativa de presas, la presa de Alloz presentaba algunas deficiencias o incógnitas importantes:

- No estaban operativos los desagües de fondo, utilizando como tales, en la medida de lo posible, los desagües intermedios, que así estaban más solicitados que lo razonable.
- Existía un nivel significativo de filtraciones en el estribo derecho, donde existen yesos.
- Las conducciones presentaban ya un envejecimiento importante.

A la vista de esta situación se decidió entonces abordar un proceso de rehabilitación en el que, manteniendo un control de las filtraciones, se llegase a reponer la posibilidad de vaciado del embalse. Es a este proceso al que se refiere esta comunicación, teniendo presente que en ese momento, la presa llevaba ya 65 años en explotación.

3 DESCRIPCIÓN DE LAS ACTUACIONES REALIZADAS

A continuación se describen cada una de las tres grandes actuaciones de rehabilitación que han permitido recuperar la operatividad del desagüe de fondo y su canal de restitución al río, así como las tomas intermedias para riegoy otras obras conexas.

3.1 *Recuperación y operatividad de los desagües de fondo*

En el acondicionamiento del desagüe de fondo se aprovechó únicamente la tubería de 1 m de diámetro embebida en el cuerpo de presa, se retiró la bola de acero que taponaba el conducto y se colocó una nueva rejilla en la embocadura.

A la salida de la tubería existente se colocó un nuevo conducto del mismo diámetro que, atravesando la galería de la antigua presa de gravedad, continúa con un tramo de 4 m de longitud, donde se producía la transición a 1.5 m de diámetro. Esta medida se mantenía en una longitud de 12.5 m hasta entrar en la caseta de válvulas, donde se produce una nueva transición, esta vez circular-rectangular que llega a la válvula de cierre, de tipo "Bureau", de 1.25×1.50 m.

Contaba esta válvula con sus correspondientes dispositivos de by-pass (300 mm) y sistema de aireación, ésta con una válvula ventosa de 300 mm de diámetro y PN 10 que se rompió en una de las pruebas, siendo sustituida por una similar pero de PN 16, colocándose además sensores y transmisores de presión para evitar futuras roturas como la ocurrida.

A continuación se colocó una válvula de regulación tipo "Howel-Bunger" de 1.5 m de diámetro.

Desde esta actuación la galería antigua ha quedado visitable, a través de la caseta de válvulas, permitiendo el acceso al paramento aguas abajo de la presa, en la salida de la tubería de desagüe.

Para disipar la energía de los caudales evacuados por el desagüe de fondo, se construyó un cuenco amortiguador del tipo impacto, con una viga en forma de L invertida de 3.65 m de altura y 0.65 m de espesor, distanciada 4 m de la boquilla de salida.

Las obras, finalizadas en 1998, incrementaron la capacidad de descarga de la máxima teórica de 15,63 m^3/s del desagüe original hasta los 21,25 m^3/s para el embalse en su nivel máximo normal.

En el año 2005, dentro del programa de revisión de elementos mecánicos de la presa y

Figura 3. Embalse vacío donde se pueden apreciar la toma de la central, el azud de desvío y la posición del desagüe de fondo.

Figura 4. Galería que atraviesa la antigua presa de gravedad.

adecuación de la misma al nuevo Reglamento de Seguridad de Presas, se cambiaron las antiguas válvulas de regulación por dos nuevas de asiento plano de diámetro 1000 mm con sus correspondientes aducciones de aire con ventosa automática y un deflector de 1000 mm de diámetro, en acero inoxidable, con un sistema de aireación. Estas válvulas resultan más adecuadas para la evacuación del tipo de agua que sale por el desagüe de fondo, con piedras y mucho fango. Previamente se sustituyó el tramo de tubería de diámetro 1,50 m por otra de diámetro 1,00 m, suavizándose el trazado con la intención de eliminar los problemas de cavitación que hasta entonces se venían presentando, debido al paso del agua de una tubería de 1,00 m a otra de 1,50 m de diámetro.

Para la realización de estos trabajos con totales garantías tanto técnicas como de seguridad fue necesaria la colocación en la embocadura del desagüe de fondo, de un tapón que pudiera soportar una carga de 70 m.c.a. Este trabajo, extremadamente especializado, fue llevado a cabo por un equipo de buzos.

Una vez finalizados los trabajos se procedió a la retirada del tapón.

Este tapón, de forma cilíndrica y con unos añadidos troncocónicos quedó también encajado en el conducto, y pese a que se generó una contrapresión para poder retirarlo, dado lo angosto de la zona de trabajo (entre la ataguía de desvío del río y el paramento de la presa), la presencia de depósitos de lodo y de materiales de todo tipo que habían sido arrastrados allí, fue necesario romperlo y sacarlo a trozos.

Con la instalación de la nueva válvula, se ha realizado la apertura de un paso circular en la viga de impacto en forma de L, así mismo ha sido necesario calcular las dimensiones del cuenco amortiguador aplicando los nuevos parámetros de la válvula.

Figura 5. Válvulas de asiento plano D = 1000 con sus ventosas automáticas.

Figura 6. Entronque del conducto para nuevo desagüe de fondo en la tubería forzada de 2.30 m de diámetro que llega a la central hidroeléctrica.

En cumplimiento del punto 4 del artículo 15 "Control de los órganos de desagüe" del Reglamento Técnico sobre Seguridad de Presas y Embalse, que dice: "En las presas de Categorías A y B los desagües profundos constarán, al menos, de dos conductos, provistos cada uno, como mínimo, de dos elementos de cierre colocados en serie", fue necesario la instalación de un segundo desagüe de fondo. Este desagüe toma el agua de la tubería forzada de diámetro 2.30 m que alimenta la central eléctrica y está situado en la margen derecha del río a unos 45 m aguas abajo del desagüe original. Se trata de un conducto de diámetro 1 m y una longitud de 50 m hasta una nueva caseta de válvulas donde se alojan una válvula compuerta de asiento plano de diámetro 1000 mm con su correspondiente by-pass y válvula ventosa para aireación y una válvula Howell-Bunger, también de 1000 mm de diámetro.

Debido a este nuevo desagüe resulta necesario la remodelación del cauce aguas abajo del mismo, ampliando el cuenco de amortiguación, demoliendo el azud existente y reponiéndolo unos 20 m más abajo.

3.2 *Acondicionamiento del Canal de Restitución del antiguo desagüe de fondo y del primer tramo de río*

La instalación de la nueva válvula de regulación del antiguo desagüe de fondo, así como la presencia del nuevo desagüe de fondo en la margen derecha, hacen necesario un replanteamiento del cauce del río Salado en esta zona.

Es necesaria la modificación del cuenco amortiguador del antiguo desagüe, la construcción de un cuenco amortiguador para el nuevo desagüe de fondo, así como el acondicionamiento del cauce del río comprendido entre ambos cuencos y entre el

Figura 7. Cuenco amortiguador con caseta de válvulas del segundo desagüe de fondo.

Figura 8. Cuenco amortiguador aguas abajo de segundo desagüe de fondo.

nuevo cuenco y el puente de la carretera de acceso a Lerate. Todo ello teniendo en cuenta que las nuevas actuaciones deben permitir contener el caudal evacuado por ambos desagües, funcionando a la vez, sin que se produzcan desbordamientos ni erosión de las márgenes.

Del estudio de las nuevas condiciones hidráulicas derivadas de la sustitución de la válvula de regulación del antiguo desagüe de fondo, se concluyó que la lámina de agua del cuenco amortiguador debía permanecer estacionaria, así como que el volumen de agua en el cuenco debía ser mayor que el existente hasta entonces, por lo que había que aumentar la profundidad de los 1.35 m que tenía a 1.75 m, así como la superficie de la lámina de agua, prolongando el cuenco longitudinalmente.

De la misma forma, como consecuencia del segundo desagüe de fondo resulta necesaria la remodelación del cauce aguas abajo, construyendo un nuevo vertedero de labio grueso, que sustituya al del antiguo cuenco y ensanchando el cauce.

Además por las condiciones de volumen y velocidad de los caudales a evacuar, se precisa adecuar la protección de las márgenes y fondo del río, entre ambos cuencos y hasta el puente de la carretera de acceso a Lerate. Para lo cual se sustituyen los materiales del lecho del río por un manto de escollera hormigonada, se aumenta la anchura del cauce así como la altura de la protección de escollera de las márgenes, macizando ésta con hormigón en masa en aquellas zonas que se conserva en todo o en parte la que ya existía.

Durante la primavera del 2007 se produjeron lluvias de extraordinaria intensidad, lo que obligó a la presa de Alloz a desaguar un importante caudal por el aliviadero de compuertas. Como consecuencia de ello, se produjeron importantes daños en el primer tramo del río Salado, entre la salida del contraembalse y el puente nuevo de la carretera de Lerate.

El monasterio de San José de Alloz se vio afectado, así como la estación de aforos existente aguas abajo del contraembalse, y los taludes del cauce, fundamentalmente de la margen derecha del río.

Como resultado de la avenida, el cauce del río disminuyó su capacidad, comprobándose mediante cálculos hidráulicos que había quedado reducida a 37 m^3/s, desbordándose el río en esta zona para caudales mayores.

Dado que en el resto del río el caudal de desbordamiento es de 75 m^3/s, resultaba necesario acometer una serie de actuaciones que intentaran acercar a esa cifra el caudal de desbordamiento en esta zona.

Para ello se hicieron una serie de estudios previos en los que se sugería demoler un puente viejo existente, redefinir el cauce en su tramo curvo, ampliarlo por la margen izquierda en el tramo recto y construir una mota lateral.

A la hora de ejecutar las obras, se comprobó que había unos puntos de estrechamiento que condicionaban toda la actuación, pues no se podían eliminar. Estos eran el punto donde se encuentra la estación de aforos y el puente nuevo sobre la carretera de Lerate. Estos condicionantes obligaron a trabajar intentando conseguir la mayor capacidad que la realidad permitía, pero manteniendo la continuidad de las líneas de flujo.

De esta manera se ha conseguido llegar a un caudal de desbordamiento de 61 m^3/s, habiéndose realizado para ello la demolición del puente antiguo citado, la reposición de la estación de aforos, recuperándose la pasarela y las pilas, y la limpieza y ensanche del cauce, colocando incluso muros de escollera en los taludes contiguos a la carretera y que habían sido seriamente erosionados.

A la vez que los trabajos de optimización hidráulica del cauce, se ha realizado una recuperación

Figura 9. Estación de aforo aguas abajo del contraembalse.

Figura 10. Muro de escollera en talud junto a carretera de Lerate.

Figura 11. Puente nuevo de la carretera de Lerate.

medioambiental de las riberas. Revegetando los taludes, tanto los que ya existían como los nuevos, plantando diferente arbolado de ribera y colocando una valla de protección de madera, de tipo rústico. Además, en la margen izquierda del río se ha uniformizado la zona entre la carretera y el borde del cauce, dejándola transitable para los paseantes. Complementándose la actuación con una limpieza de ramas en ambos márgenes del río.

3.3 *Recuperación de las tomas intermedias de riego*

Durante la avenida del año 2007, ya citada, se produjeron daños de diferente importancia en los desagües intermedios de la presa. Éstos, que constituyen la toma de riego, nunca se han utilizado como tal, siendo la central hidroeléctrica la que, con el caudal que pasa por sus turbinas, abastece a las necesidades del riego de la zona.

En otras obras de emergencia realizadas en el año 2005, fue reparado el conducto izquierdo de la toma de riego que presentaba pequeñas perforaciones por oxidación en el carrete intermedio. Esta reparación consistió en la colocación de una tubería de acero inoxidable de 8 mm de espesor y 975 mm de diámetro, en el interior de la tubería existente, en la posterior inyección de mortero resinoso entre ambas, así como en la sustitución de los carretes, en la colocación de un nuevo by-pass y una nueva aducción de aire (con ventosa y válvula compuerta para el cierre de seguridad).

A finales del 2008 finalizaron las actuaciones en los conductos de la toma de riego.

A la cámara de maniobra se accede a través de la caseta de acceso que se encuentra junto al vano izquierdo del aliviadero y se desciende mediante una escalera metálica.

La cámara se encuentra debajo del aliviadero, y al bajar el embalse para la reparación de la toma de riego, tras la avenida de 2007, se comprobó que estaba descalzada, siendo necesaria la realización de un tratamiento de recalce de la misma.

En los conductos central y derecho, como ya se hizo en el conducto izquierdo, se colocó una tubería interior de acero inoxidable de diámetro 975 mm, desde la embocadura hasta la válvula compuerta, inyectando con mortero resinoso el

Figura 12. Cámara de válvulas de la toma de riego.

Figura 13. Central oleohidráulica de las válvulas de la toma de riego.

Figura 14. Cuadro de mando y control de las válvulas de la toma de riego.

espacio intermedio, posteriormente se sustituyeron los carretes por otros, y se colocaron nuevos by-pass y aducciones de aire. Todos los elementos sustituidos son de acero inoxidable AISI-304 y las bridas de acero al carbono.

En cuanto a las válvulas compuerta, fueron desmontadas y revisados todos y cada uno de sus elementos, sustituyéndose aquellos que se encontraban deteriorados. A las partes estructurales de las válvulas se les realizó un tratamiento de chorreado y posterior pintado con pintura de poliuretano. Se procedió, además, al cambio de la tornillería por otra nueva.

Finalmente indicar, que se ha realizado la motorización de los equipos mediante cilindros oleohidráulicos, con el montaje de un grupo hidráulico y un nuevo cuadro de control y mando.

Se consigue así dejar operativa la toma de riego como una toma intermedia para la regulación de la presa de Alloz.

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Presa de Villagatón. Impermeabilización, drenaje y auscultación

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ABSTRACT: Villagatón's dam is an arch—gravity of 39 meters of height on foundations. His construction ended in 1995; in spite of that the dam has not entered normal exploitation, yes it has had several fillings, coming practically to 100%. The behavior of the dam has been an anomalous partner with filtrations of certain importance, though never related to lack of structural safety. The fact of not having entered exploitation has helped not to have better information across the system of monitoring, for what the structural behavior is only partially an acquaintance. Before this situation, in 2009 there were carried out actions related to the improvement of the screen of waterproofing, reinforcement of the drainage network, as well as improvement and extension of the system of monitoring. The obtained results are still partial and with scanty information of the devices of control; the filtrations have diminished significantly.

1 INTRODUCCIÓN

La presa de Villagatón se ubica en el término municipal del mismo nombre, en provincia de León. Se sitúa sobre el Arroyo de la Vega, reteniendo un embalse de 4 hm^3, con un uso principal dedicado al riego, aunque la presa carece actualmente de usuarios.

La presa es un arco-gravedad de 39 metros de altura sobre cimientos y 35 metros sobre el cauce, con 189 metros de longitud de coronación, a la cota 1.055. Dispone de una galería perimetral y otra horizontal 14,50 metros por debajo del nivel de coronación. El paramento de aguas arriba es vertical y el de aguas abajo con pendiente ¼. Las dimensiones en planta de la presa se definen mediante arcos de tres centros; en el eje de definición de la presa dicho arco tiene un radio de 70 metros, con radios laterales de 140 metros. La presa dispone de juntas verticales radiales que dividen las 13 ménsulas de la estructura.

La cimentación está formada por pizarras negras de grano fino del Ordovícico Superior, poco alteradas en superficie, con presencia de óxidos y mineralizaciones de pirita. No existe ninguna falla que afecte al área de la cerrada; por su parte, las dos laderas del macizo están afectadas por tres familias de diaclasas. La presa no se encuentra ubicada en zona sísmica.

Su construcción se inició en el año 1991, finalizando los trabajos en 1995. En este mismo año se llevó a cabo un tratamiento de consolidación, impermeabilización, drenaje e inyección de juntas en la presa.

La puesta en carga de la presa se inició a principios 1997 y se prolongó hasta mediados del año 2000, registrándose filtraciones notables, provenientes del contacto hormigón-roca, determinadas zonas de hormigón de peor calidad y a través de algunas de las juntas verticales y de tongada.

Aunque la presa no ha entrado en explotación normal por no tener todavía dispuesta la infraestructura de distribución de agua, sí ha tenido varios llenados, llegando prácticamente al 100%.

Figura 1. Vista del paramento de aguas abajo húmedo por precipitaciones, con embalse vacío, antes del inicio de los trabajos.

2 ANTECEDENTES: CONSTRUCCIÓN Y PUESTA EN CARGA

Aún perdiendo muchos detalles, el desarrollo de la obra puede resumirse de la siguiente forma:

- El 2 de mayo de 1991 se iniciaron los trabajos en la presa en todas las unidades de obra que no se veían afectadas por las exigencias de un informe de Vigilancia de Presas. Una vez redactado el modificado, se continuaron las obras.
- Entre mayo y junio de 1992 se realizan las excavaciones de la cimentación y en julio de ese mismo año se inicia el hormigonado del zócalo.
- En septiembre de 1992, durante el hormigonado de diferentes bloques del cuerpo de presa, se detecta en las probetas de ensayo una bajada en la resistencia a la compresión, por lo que se paraliza el hormigonado del cuerpo de presa. Se redactó un informe y se detectaron problemas en la calidad de la arena empleada, así como en el cemento.
- La interrupción de la obra se prolongó hasta julio de 1993, fecha en la que se reanudaron los trabajos. El hormigonado finalizó en diciembre de 1993.
- En 1995 finalizan los trabajos de construcción de la estructura de cierre.

El archivo técnico de la presa dispone de diversos estudios, proyectos e informes que han aportado datos y circunstancias de gran interés sobre el estado de la presa y su cimiento. Algunos de estos informes, redactados desde Vigilancia de Presas por técnicos de reconocido prestigio, llevaron a modificar la solución inicialmente prevista (modificación del proyecto original y proyecto complementario), así como a modificar las condiciones de puesta en obra del hormigón, que ya se detectó que podría quedar con excesiva porosidad, con posibles repercusiones de cara al envejecimiento. Se indican a continuación algunos de ellos, añadiendo algunos comentarios de interés.

- *Informe de Guillermo Gómez Laá. Abril de 1990*. "*La comprobación tensional de la bóveda da compresiones de 38 kg/cm^2 y tracciones máximas de 6 kg/cm^2, habida cuenta del peso propio, empuje hidrostático y efecto térmico; no se tiene en cuenta la subpresión, previéndose, en cambio una red de drenaje bien concebida*". Después debió venirse a menos la previsión. Se propone se estudie bien la procedencia de los áridos; también la mejora del cauce hasta pasado el pueblo para lograr 25 m^3/s, así como contactar con Vigilancia de Presas para temas de auscultación, tratamiento y sellado de juntas, y modificaciones en los estribos.
- *Informe de Guillermo Gómez Laá. Septiembre de 1992*. Se recoge la nueva excavación que mejora la incidencia de los arcos, según se proponía en el informe anterior de Vigilancia. "*La cimentación se estima impermeable y estable, aunque necesite bulonados y mallazo durante la fase de construcción. Hay una veta subvertical de caolín en la parte alta del estribo derecho, sin mayor dificultad que llaguear y rellenar de hormigón*". Inmediatamente antes de las conclusiones se dice "*es fundamental la pantalla de drenaje, perforada de forma que afecte el mayor número de diaclasas; necesarias las inyecciones superficiales de consolidación, con probable admisión reducida, dado el cuidado con que se ha excavado el cimiento; la pantalla de inyección profunda, probablemente no será necesaria*".
- *Informe de Moisés Rubín de Célix Caballero. Marzo de 1993*. "*Se anota una porosidad del mortero alta (17%) y del hormigón (11%) consecuente con la del mortero, ambos valores influyen en el resto de características del hormigón. La unión entre tongadas se ha mostrado deficiente por falta de tratamiento adecuado de la junta fría y por falta de mortero de retoma. Se recomienda estudiar la dosificación para mejorar la permeabilidad; tratar los hormigones ya puestos para limpiar su paramentos e impermeabilizarlos por aguas arriba y por aguas abajo (para evitar la percolación); limpiar muy bien y humedecer las superficies horizontales de tongada antes de colocar el siguiente hormigón; utilizar un mortero de retoma con tamaño máximo 25 mm y espesor entre 3 y 5 cm; el curado de la tongada con agua debe prolongarse hasta el hormigonado de la superior; interesa poner varillas extensométricas para auscultar los bloques hormigonados hasta la fecha*". Hay un gráfico que marca claramente las tongadas previas a la nota emitida, en general, todo el contacto con la cimentación, precisamente donde se han concentrado buena parte de las filtraciones existentes en la presa hasta ahora.
- *Informe de Moisés Rubín de Célix Caballero. Diciembre de 1993*. Con la obra a falta de 600 m^3 de un total de 32.500 m^3. "*Se establecen recomendaciones para el tratamiento de consolidación (hasta unos 5 m. de profundidad). Ya se propone estudiar la permeabilidad entre juntas horizontales inyectándolas, si el preciso, con microcemento o resinas epoxídicas. Se aportan cálculos que indican existencia de tracciones aguas arriba y hace suponer escasa resistencia de las ménsulas, que trasvasarán su carga a los arcos. Por ello, se dice, es imprescindible el lograr una buena inyección de juntas verticales. Se proponen unas inyecciones en abanicos en los estribos para reforzar la transmisión de esfuerzos del arco superior. Se proponen extensómetros de tres varillas, de forma que una quede dentro del hormigón y proporcione información sobre aperturas de juntas entre tongada*

(esta información también la darán los grupos de seis extensómetros instalados aguas arriba, y con mucha fidelidad, aunque puntual, al haber sólo dos). *Se proponen tres secciones con péndulos: central, y dos laterales* (la forma de la presa y su entidad lo merecían). *Advierte de la importancia de situar la inyección de juntas en su época adecuada desde el punto de vista térmico*".

– *Impermeabilización, consolidación y drenaje*. En 1995 se llevó a cabo un tratamiento de consolidación, impermeabilización, drenaje y tratamiento de juntas en la presa de Villagatón.

  Consolidación: 54 taladros, 546 metros (entre 7 y 24 metros de profundidad), 9,415 toneladas de cemento y 17,22 kg/ml de taladro. Los taladros tuvieron 12º hacia aguas arriba y 35º hacia aguas abajo.

  Abanicos m.d.: 8 taladros, 102,5 metros 1,600 toneladas de cemento y 15, 6 kg/ml.

  Abanicos m.i.: 4 taladros, 48 metros 0,900 toneladas de cemento y 18,75 kg/ml.

  Drenes del cimiento: 11 taladros verticales, 190 metros, 12 metros de separación entre taladros (muy poco).

  En base a los datos obtenidos con el tratamiento de consolidación y drenaje, se confirma la no necesidad, en principio, de ejecutar la pantalla de impermeabilización. El problema de subpresiones, se creía inicialmente resuelto con el tratamiento de drenaje.

– *Sellado de juntas*. La inyección de juntas se inició el 3–04–95 y se terminó el 18–05–95. Se emplearon dosificación de 1/1 en peso y se utilizaron presiones de cierre de 4–5 kg/cm^2. En muchas juntas de coronación fue necesario rehacer los conductos de inyección.

  Parece que las juntas verticales de construcción no quedaron impermeables en su parte baja; también se duda de si quedaron bien inyectadas en su parte alta donde típicamente se llega ya con la presa caliente y se efectúa un relleno pobre, todo dicho desde el punto de vista estructural, no de impermeabilización. Si se confirma en detalle la información de que los cambios térmicos se propagan a toda la masa del hormigón en uno o dos meses, la juntas habrán sido inyectadas ya algo cerradas en su parte alta y estarán abriendo todos los inviernos. La opción de inyectar con lechada de cemento practicando nuevos taladros es de dudosa realización práctica, pudiendo llegarse a que "la lechada no entra o entra muy poco". Es necesario saber si hay goma (water stop) en la junta, en su intersección contra la cimentación y probablemente convenga pensar en inyecciones de resina epoxi para esta impermeabilización. Una vez que ahora se pretendía trabajar en las juntas, parecía muy conveniente conocer el estado en que estaban antes del inicio de los trabajos, 14 años después de su inyección, lo que podría haberse deducido de los medidores tridimensionales, en caso de se hubiera dispuesto de suficientes campañas de datos.

– *Primera puesta en carga*. Se realizó la primera puesta en carga entre enero de 1997 y mayo del 2000, mes en que se alcanzó la cota 1053. Se desarrolló con tres escalones a las cotas 1035, 1045 y 1053, concluyendo el equipo encargado del control de la presa que "*no se detectaron anomalías en el comportamiento de la presa*". Sin embargo, las filtraciones alcanzaron los valores de 1,17 l/s, 11,2 l/s y 12,84 l/s en cada uno de los escalones, respectivamente. Por su parte, los piezómetros captaron presiones que, en porcentaje sobre el nivel de embalse en cada una de las fases, llegaron a 83, 63 y 62%. Cabe sin embargo calificar las filtraciones detectadas de importantes y la subpresión de elevada, sobre todo si se tiene en cuenta la excelente calidad del macizo rocoso en la cerrada, salvo por el potencial ataque que puede sufrir el hormigón por aguas ácidas.

– *Informe de Moisés Rubín de Célix Caballero. Marzo de 1998. La interrupción del hormigonado, por diferencias entre las resistencias pedidas en el pliego y las obtenidas, se extendió entre octubre de 1992 y julio de 1993. En febrero de 1997 se procedió al replanteo de la auscultación complementaria. El nivel del embalse se encontraba, por esas fechas, a la cota 1035.50 (NMN = 1.053,00); se aprecian humedades en el paramento de aguas abajo de la presa, en cota inferiores de la margen izquierda y una filtración a través de cimiento, en la misma margen, de 2 l/min. En la galería perimetral, la filtración era de unos 100 l/min. En un informe sobre el segundo escalón se recogen 60 puntos de filtración en las galerías. En la página 23, indica que cuatro piezómetros no pueden captar la realidad de las presiones en la cimentación y que la red de drenaje es escasa* (con lo que se coincide). *También cita la importancia, despreciada hasta entonces, de analizar los datos de los extensómetros de varilla. Aboga después por el interés de la información proporcionada por los extensómetros de hormigón. Propone una campaña de inyecciones para corregir filtraciones, hechas con un nivel intermedio y comprobadas subiendo el nivel con el equipo de inyección todavía en obra, por si hubiera que acometer nuevas inyecciones complementarias. Se propone mejorar la red de drenaje; también equipar la presa con mayor número de piezómetros. En la conclusión cabe destacar la 6ª, en pro de una adecuada interpretación del comportamiento de la presa, dejando de lado modelos muy poco reales. En la 8ª comenta el interés de interpretar los extensómetros de hormigón para caracterizarlo.* No se puede olvidar el subrayar

la conclusión 9ª del citado informe sobre la caracterización mineralógica de la cimentación (ya se ha dicho antes que las pizarras con pirita desarrollan alteraciones generando un ambiente ácido, pH bajo, que favorece el ataque al hormigón).

– *Actuaciones recomendadas en el zócalo*. En el documento "Remate de zócalo de la presa en ambas márgenes" elaborado por Tragsa con fecha de enero de 2008, se cita la conveniencia de proteger la roca del pie de aguas abajo para evitar que se degrade "*pudiendo provocar daños en el cuerpo de presa*". Por la experiencia habida en otras presas de hormigón cimentadas en pizarras y con presencia de piritas, se estima esta recomendación como muy acertada, incluso convendría ampliar en lo posible esta protección frente a aguas de lluvia, principalmente evitando su acceso a las juntas desde coronación.

3 PROBLEMÁTICA EXISTENTE

El comportamiento de la presa ha sido anómalo en algunos aspectos, nunca relacionados con falta de seguridad estructural. Cabe destacar la presencia de filtraciones, casi de forma general, en el contacto de cimentación, que adquirían cierta importancia en la margen derecha y en la parte alta de los estribos, también un número limitado de filtraciones por juntas de tongada y juntas verticales de construcción.

Probablemente la falta mayor continuidad en la explotación ha contribuido a que el sistema de auscultación no haya sido explotado de forma completa y continuada y, por ello, el comportamiento estructural sea solo parcialmente conocido.

Figura 2. Vista de la escalera de galería perimetral por margen derecha, con anterioridad al inicio de los trabajos. Nótese la necesidad de doble recogida de filtraciones, aguas arriba y aguas abajo. Embalse aproximadamente al 50%.

Las circunstancias anteriores han impedido el mantenimiento de niveles altos de embalse. Las filtraciones que han llegado a registrarse en los aforadores próximos a la cámara de válvulas, en el tramo horizontal de galería perimetral, han sido de 25 l/s en margen derecha y 15 l/s en margen izquierda, lo que equivale a un total de 2.400 l/min.

Distintos técnicos han estudiado el problema en la presa de Villagatón, proponiendo diferentes tratamientos, que pueden resumirse de la siguiente forma:

– Eliminación de filtraciones en el contacto presa cimentación.
– Tratamiento de impermeabilización en las juntas horizontales. Se refiere a juntas entre tongadas que dan agua en el paramento de aguas abajo.
– Tratamiento de coqueras y nidos de áridos existentes. Proponen utilizar resinas como alternativa, lo que puede ser una buena solución práctica.
– Abanicos de impermeabilización en los estribos para impedir la entrada de agua de lluvias.
– Restitución de la red de drenaje tras las inyecciones ejecutadas, mejorando si cabe la situación de partida.
– Emplear resinas para el tratamiento de filtraciones por el hormigón y para la impermeabilización de juntas verticales.

Como añadido, el impedir cualquier entrada de agua de las laderas hacia el interior de la cimentación es del todo recomendable, para evitar el ataque de aguas ácidas al hormigón de la cimentación. Por el tipo de cimentación, es de sospechar que las filtraciones vengan por las proximidades del

Figura 3. Zócalo de aguas abajo sin rematar en margen izquierda, susceptible de entrada de aguas ácidas.

contacto; a falta de mayor información, convendría realizar algún taladro de reconocimiento para averiguar si hay filtraciones más profundas.

Por otro lado, a la hora de decidir sobre la mejor solución para la restitución de la red de drenaje en el cimiento, convendría fijarse en las subpresiones existentes, atendiendo a los piezómetros, y realizar más taladros (más juntos) cuando éstas sean altas.

4 SOLUCIÓN ADOPTADA EN VILLAGATÓN

La información aportada por el archivo técnico de la presa, rica en matices, ha servido para analizar la problemática existente y su génesis. La elección de una solución adecuada para el problema en la presa de Villagatón y su cimiento se ha apoyado, por un lado, en el análisis de la documentación disponible, de otro, en las inspecciones visuales efectuadas. Por desgracia no se ha podido sacar mucho partido al sistema de auscultación implantado en origen en la presa, por no haberse atendido convenientemente, siendo las campañas de lecturas muy escasas.

Con los antecedentes citados, y basándose en la experiencia acumulada en otras presas, se ha acometido un proyecto de rehabilitación que contempla, principalmente, una mejora de la pantalla de inyección, incluida la superficie de contacto, un refuerzo de la red de drenaje, unas inyecciones puntuales en fugas por juntas verticales y de tongada y una puesta a punto y complementación del sistema de auscultación. Sin ánimo de entrar en profundidad, la metodología para las perforaciones e inyecciones, ángulos de ataque, diámetros y materiales a emplear, han sido las siguientes, indicando algunas modificaciones sobre el proyecto inicialmente previsto:

- Ejecución de un número limitado de taladros con extracción de testigo a modo de comprobaciones puntuales del contacto presa—cimiento y juntas verticales.
- Reducir a 3 m. la distancia máxima entre taladros de inyección de consolidación, con una profundidad de unos 5 m. por debajo del contacto presa—cimiento.
- Variar el ángulo respecto a la vertical de los taladros de inyección de consolidación, pasando a ser de 10º hacia aguas arriba y 5º hacia aguas abajo dispuestos de forma alternativa.
- Obturar a distintas profundidades para tener un conocimiento de qué zonas son las que demandan mayor cantidad de lechada durante la inyección.
- Necesidad de proteger el pie de aguas abajo de la presa frente al agua de lluvia mediante un zócalo de hormigón.

Figura 4. Parte de un testigo de reconocimiento del cimiento, en el contacto roca—hormigón, extraído con carácter previo al proceso de inyección.

- Complementar las inyecciones efectuadas en aquellas zonas que han presentado admisiones considerables de lechada, con taladros adicionales ubicados convenientemente.
- En caso de inyectar las juntas verticales, realizarlo a través de taladros nuevos que intersecten la junta una vez salvada la water—stop, empleando preferiblemente resinas como material de relleno.
- Conveniencia de comprobar las inyecciones de consolidación ejecutadas.

En cuanto a la posibilidad de efectuar cálculos mediante elementos finitos, se debe ser escéptico sobre los resultados esperables dada la complejidad de la estructura real, que difícilmente cabe en las simplificaciones que necesita el programa. No podrán obtenerse resultados aceptables en tanto no se disponga de un registro de histórico, suficientemente amplio y de calidad, a través de los datos de auscultación que permita calibrar un modelo matemático.

5 ACTUACIONES PARALELAS

En paralelo a las actuaciones encaminadas a la mejora de las condiciones en que presta servicio la presa de Villagatón y su cimiento, se han venido desarrollando otros trabajos, que de forma resumida han consistido en:

- Remate del zócalo de la presa en ambas márgenes. La unión del hormigón y la roca al pie de la presa, en ambas márgenes se realizó de manera

descuidada y sin un buen remate. Se ha ejecutado una pequeña acequia para encauzar las aguas de escorrentía que discurren por ambos zócalos y dirigirlas hacia el cuenco amortiguador. Con ella además se tapará la junta entre roca y hormigón y se protegerá la roca para evitar que se degrade, pudiendo provocar daños en el cuerpo de presa.

– Reconstrucción del cuenco amortiguador y del contradique. El cuenco amortiguador sufrió desperfectos durante la crecida acaecida en el invierno del 2000–2001. En este episodio, el muro de la margen izquierda del cuenco, formado por escollera se vino abajo, quedando inutilizado el camino de acceso al pie de la presa para vehículos. El contradique también había sufrido desperfectos, y parte de la escollera de protección de la margen derecha aguas abajo del contradique también deslizó, viniéndose abajo. Se ha acondicionado el cuenco amortiguador, mejorando el muro de protección de la margen izquierda con un muro de hormigón armado; también se ha reconstruido el contradique y las escolleras de protección, respetando las dimensiones calculadas en el proyecto complementario.
– Vaciado del colchón amortiguador. Se ha dispuesto una arqueta en el estribo derecho que permite la disposición de una válvula de mariposa de 400 mm. de diámetro para el vaciado del colchón amortiguador de cara a labores de limpieza.
– Reacondicionamiento de las galerías interiores. Además de acondicionar el canal de desagüe, se han introducido en las galerías interiores una serie de mejoras para garantizar un acceso rápido y sin problemas a las salas de válvulas y de instrumentación de auscultación.

Figura 5. Reconstrucción del cuenco amortiguador: muro de hormigón armado en margen izquierda, reconstrucción de contradique y reposición de escollera de protección.

– Instalación de un grupo electrógeno en una caseta prefabricada, necesario para que el funcionamiento de la iluminación interna de las galerías, las válvulas electromotorizadas y los elementos de auscultación no se interrumpa en caso de un corte accidental del suministro eléctrico.
– Construcción de una escalera exterior en margen izquierda.
– Acondicionamiento de la casa de administración.
– Acondicionamiento de los accesos a la presa.

6 ACONDICIONAMIENTO Y MEJORA DEL SISTEMA DE AUSCULTACIÓN

Tras las visitas a la presa, con carácter previo al inicio de los trabajos de acondicionamiento, se resumieron los aspectos que debían ser atendidos dentro del sistema de auscultación de la presa, proponiendo para ello las actuaciones oportunas. Éstas fueron establecidas en función de:

– Estado en que se encontraban dispositivos de auscultación instalados.
– Posibilidad de recuperación de dichos los equipos.
– Ampliación del sistema de auscultación para cubrir posibles carencias.
– Automatización de todos aquellos equipos que lo permitan.
– Disposición de sistemas de comunicaciones y herramientas informáticas acordes a la técnica actual, que pueda cubrir los requerimientos de los trabajos de explotación.
– Suministro de equipos de medida para reiniciar cuanto antes las campañas de toma de datos.

Para el acondicionamiento y mejora del sistema de auscultación se establecieron las siguientes actuaciones, cuya finalización está prevista en el verano de 2010. En todos aquellos dispositivos que lo permitían, se ha previsto la conexión a una unidad de lectura automática manteniendo la posibilidad de realizar lecturas manuales, así como crear archivos de lectura con sus gráficos para la interpretación:

– *Extensómetros de hormigón*. Existían 4 grupos unidimensionales en las proximidades de coronación, situado el activo en la dirección de la tangente al arco, a 1,5 metros. del paramento de aguas arriba; disponían de corrector. También se instalaron dos grupos de seis extensómetros junto al pie de aguas arriba de dos bloques, así como a 1,5 metros del paramento de aguas arriba. Se ha revisado el funcionamiento de cada aparato realizando lecturas con una unidad portátil.

– *Piezómetros de cuerda vibrante*. Se disponía únicamente de 4 piezómetros en cuatro bloques altos centrales, situados dos metros pasado el contacto roca-cimiento. Por la poca información captada del seguimiento del primer llenado se intuía que las presiones no son despreciables, más bien al contrario, por lo que resultaba necesario colocar piezómetros más aguas arriba y alguno más, para captar una ley de subpresiones en cimentación y un gradiente de pérdida de presión a medida que se camina hacia aguas abajo. Al acometer unas inyecciones y un nuevo drenaje, hay una ocasión buena de ubicar más piezómetros (ampliar las cuatro secciones previstas a unas ocho) para controlar mejor la cimentación.
– *Extensómetros de varilla*. Existían cuatro extensómetros de varilla de dos varillas cada uno, estando la varilla corta anclada en el hormigón y la larga a cuatro metros por debajo de la cimentación. Están reunidos en dos secciones, con un extensómetro vertical y otro inclinado 45º hacia aguas abajo. Existían al menos otros cuatro extensómetros de varilla que controlaban la cimentación aguas abajo de la presa, todos en el tacón de hormigón de aguas abajo. Se ha suministrado un comparador adecuado para realizar las lecturas, preparado archivos de lectura y dotado a las varillas de sensores eléctricos conectados a una unidad de lectura automática. También se han instalado nuevos extensómetros, dirigiendo un conjunto hacia aguas arriba para captar desplazamientos de la estructura hacia aguas abajo y aperturas en el pie de aguas arriba.
– *Péndulos*. La presa disponía de una alineación directo + invertido en el bloque 1, estando el anclaje de cimentación 15 m. bajo el contacto. Se ha previsto revisar coordinómetros y equipos de lectura eléctrica actuales, sustituirlos en caso necesario, realizar lecturas de reinicio de campañas, preparar archivos de lectura. Una vez asegurados unos coordinómetros manuales fiables, se han dispuesto también coordinómetros ópticos, conectados a una unidad de lectura automática.
– *Termómetros*. Se disponía de un total de 64 termómetros del tipo PT100. Se han revisado, proporcionado unidad de lectura y creado la base de datos.
– *Nivelación*. Existían quince clavos de nivelación, 11 en la coronación y 4 en la ladera de la margen izquierda. Se ha comprobado el buen estado del sistema de nivelación.
– *Medidores externos de juntas*. Se disponía de un total de 18 medidores de juntas tridimensionales, instalados siempre en el hastial de aguas arriba de la galería. Se ha comprobado el buen estado de cada medidor y suministrado al inicio de los trabajos un comparador para la toma de lecturas y preparado archivos de la base de datos.
– *Aforadores*. Se habían dispuesto en origen dos aforadores en V tipo Thompson, de h = 0,20, que se han sustituido por otros de mayor tamaño (h = 0,30). Uno recoge margen izquierda y otro margen derecha. Están dotados de sensor ultrasónico de 4–20 mA.
– *Colimación*. Existían cuatro alineaciones en coronación para controlar un total de 10 puntos. Se ha comprobado el buen estado del sistema de colimación y repuesto las bases deterioradas.
– *Limnímetro y escala*. La presa no dispone de estos elementos, que resultan imprescindibles en la explotación, por lo que se ha instalado un limnímetro con toma directa al embalse y una escala de nivel adosada al paramento de aguas arriba.

Figura 6. Cabezas de extensómetros de varilla, automatizados, de nueva implantación.

Figura 7. Medidores tridimensionales instalados en la presa en origen. Inmediatamente por encima y por debajo, conductos antiguos de inyección y purga de junta vertical.

– Otros. acelerómetro, estación meteorológica, equipos de medida, centrales de lectura, posicionamiento de compuerta, fibra óptica.

Con el Sistema de Auscultación establecido se podrá realizar un seguimiento del comportamiento de la presa de Villagatón acorde con la entidad de la estructura de cierre y de las características de su cimiento. Una vez sea implantado, incluyendo el acondicionamiento de los dispositivos actualmente existentes, se encontrará en condiciones de obtener un historial mínimo de campañas de datos que permitan redactar un informe de comportamiento, para lo cual resultará conveniente disponer de aproximadamente un año de campañas de toma de datos.

Convendrá igualmente establecer una sistemática de redacción de informes anuales de comportamiento, por parte de especialistas en la materia, al menos en los primeros años, pudiendo prorrogarse por períodos de tiempo más amplios si se considera conveniente.

7 RESULTADOS OBTENIDOS, AÚN PARCIALES

Para ejecutar los trabajos descritos y debido a la entidad de las filtraciones y material a emplear en la inyección, fue necesario el vaciado total y controlado del embalse, para lo que no hubo mayor problema al no existir condicionantes medioambientales, hidráulicos o usuarios que potencialmente pudieran haberse visto perjudicados.

En una primera actuación se han inyectado un total de 202 taladros con una longitud media próxima a los 10 metros. La cantidad total de cemento empleado se aproximó a las 34 toneladas, con una admisión media en el total de los taladros de 16,9 kg/m.l. (relación a/c empleada de aproximadamente 2/3). La inyección se efectuó de tres veces, obturando de la siguiente forma:

– Un metro por debajo del contacto hormigón-roca, inyectando por lo tanto la longitud restante hasta el fondo del taladro, con toda la perforación en roca. Presión de trabajo 5 kg/cm^2. Admisión media de 7,9 kg/m.l. de taladro (total 8.066 kg cemento).
– Un metro por encima del contacto hormigón-roca. Inyectando por tanto un metro por encima y por debajo del contacto de cimentación, precisamente la zona sospechosa de acaparar la mayor parte de las filtraciones registradas. La perforación inyectada se reparte por igual en roca y hormigón. Presión de trabajo 3 kg/cm^2. Admisión media de 37,3 kg/m.l. de taladro (total 15.088 kg cemento).
– En boca de inyección. Último tramo inyectado en cada taladro, todo en hormigón en este caso. Presión de trabajo 3 kg/cm^2. Admisión media de 18,4 kg/m.l. de taladro (total 10.672 kg cemento).

Las comunicaciones en las inyecciones de los taladros fueron muy limitadas en número; apenas cinco de ellos comunicaron con la zona de aguas arriba de la presa y uno sólo con la galería.

Los resultados parciales obtenidos con la anterior campaña de inyecciones corroboran la visión del problema de buena parte de los técnicos que han pasado por la presa de Villagatón, entre los que se encuentra el personal técnico de ATI.

Al menos son tres las primeras conclusiones que pueden extraerse:

– No es necesario complementar la pantalla de impermeabilización gracias a la impermeabilidad mostrada por el macizo rocoso de la cerrada, formada por pizarras negras de grano fino.
– El contacto hormigón-roca no quedó bien resuelto en la obra, mostrando gran facilidad a la penetración del agua proveniente del embalse y/o de lluvia. Esta circunstancia podría acarrear graves problemas en el cimiento a medio—largo plazo al verse atacado el hormigón de cotas bajas por aguas ácidas, con el inconveniente añadido de tener una renovación importante.
– El hormigón resultante de las primeras tongadas de la presa es de baja calidad, con numerosas coqueras y juntas de hormigonado mal tratadas, que han hecho que el conjunto de la obra se haya mostrado muy poroso en sus primeros metros, tal y como ya se advertía que podía ocurrir en los informes de Vigilancia de Presas de 1990 a 1993. Tampoco es éste un problema exclusivo al caso de Villagatón, ya que en las presas es frecuente el cambio en la dosificación del hormigón y mejora de la puesta en obra conforme avanzan los trabajos.

Por su parte, se ejecutó una nueva red de drenaje que podrá complementarse en aquellas zonas que reflejen elevadas presiones a través de los piezómetros de cuerda vibrante de reciente instalación. Se dispusieron un mínimo de dos drenes por bloque, con una profundidad de 20 metros por debajo del contacto hormigón-roca, y una longitud total ejecutada de 763,8 metros.

El resultado de la campaña de inyecciones y posterior ejecución de red de drenaje se ha podido comprobar con el embalse aproximadamente al 60%, pudiendo calificarse de satisfactorio, habiéndose corregido buena parte de las filtraciones que aparecían en la presa en anteriores ocasiones con cotas similares en el embalse y misma época del año. Aunque aún no se dispone de datos de los aforos, con una simple inspección visual ya

Figura 8. Zona de galería perimetral que presenta mayores filtraciones tras las actuaciones acometidas. Las canaletas de recogida de filtraciones, ahora sí son capaces de conducir el agua filtrado hacia el nivel inferior y después al exterior, no sólo por haberse reducido el caudal, sino también por el acondicionamiento de las galerías. Será una de las zonas susceptibles de un segundo tratamiento, esta vez más localizado.

Figura 9. Zócalo de aguas abajo en margen izquierda una vez rematado mediante hormigonado y formación de cacera de recogida de aguas pluviales. Juntas de tongada con filtraciones apreciables, con formación de hielo debido a las bajas temperaturas reinantes.

se puede evaluar lo acertado de las actuaciones desarrolladas. Sin embargo, a pesar de los buenos resultados obtenidos, se han localizado zonas puntuales que aún presentan aportes de agua, muy probablemente relacionadas con hormigón defectuoso, que se refleja en las galerías a través de coqueras y juntas frías mal tratadas.

La inyección de juntas verticales ha tenido que ser desfasada de la impermeabilización y drenaje para, por un lado, notar su verdadera magnitud y, de otro, llevarla a una época del año fría, con las juntas abiertas. Podrá efectuarse una comprobación posterior al tratamiento específico de cimentación, todo ello para localizar aquellas juntas que puedan presentar filtraciones significativas. Una vez iniciadas las campañas de lecturas a través de los medidores tridimensionales, se ha tratado de realizar un análisis sobre la conveniencia, o no, de llevar a cabo una re-inyección de juntas parcial o generalizada en la presa. Sin embargo, debido a las pocas campañas disponibles y estar a punto de finalizar el período óptimo para efectuar este tipo de trabajos (la presa se encuentra ya muy caliente), se ha pospuesto la decisión hasta disponer de campañas suficientes del comportamiento de las juntas verticales de presa.

Una vez subsanado el problema de las filtraciones a través del contacto del cimiento, ha quedado un número reducido de zonas localizadas pendientes de un tratamiento posterior:

– Hormigón o contacto con roca.
– Juntas verticales.
– Juntas frías de tongada.

Convendrá efectuar estos tratamientos localizados con inyecciones de resina, pudiendo comprobarse su efectividad con niveles altos de embalse. Al no esperarse grandes admisiones, podrá asumirse el sobrecoste que supone el empleo de este tipo de material frente a la lechada convencional, especialmente indicado para las juntas, tanto verticales como de tongada, con elevado control en la puesta en obra. En este sentido, si alguno de los drenes ejecutados recientemente se viera afectado por nuevas inyecciones, deberá ser reperforado o complementado con otro nuevo, aunque por suerte las zonas susceptibles de tratamiento quedan suficientemente alejadas de los drenes.

En paralelo con el desarrollo de los trabajos, ya se ha indicado la protección efectuada sobre el pie de aguas abajo, con un recubrimiento de hormigón para limitar, en lo posible, futuros ataques al hormigón de la presa por aguas ácidas de escorrentía o de filtraciones. Una segunda actuación parcial, ha desembocado en una impermeabilización especial de la coronación, particularmente de las juntas que afloran a ella.

8 CONCLUSIONES

Las actuaciones desarrolladas en la presa de Villagatón, sin duda, van a mejorar las condiciones en que presta servicio la estructura de cierre y su cimiento. Se ha corregido el principal problema, elevadas filtraciones a través del contacto presa—cimiento, a la vez que se han protegido en el hormigón zonas sensibles de recibir aguas ácidas que puedan afectar a sus características a medio o largo plazo.

La reinyección de juntas verticales de presa de forma generalizada probablemente no deba acometerse como tal, sino que, una vez puesta en carga total o parcialmente de la presa, podrá dictaminarse sobre la necesidad de un tratamiento puntual, así como el material más adecuado para ello, probablemente resinas frente a lechadas de cemento o microcemento. Las circunstancias actuales de la presa, con juntas muy cerradas por efecto térmico y presa vacía, desaconsejan afrontar cualquier tipo de reinyección de juntas verticales, ya que las admisiones serían muy limitadas, inyectándose poco más que el conducto de inyección. Además, también resulta del todo recomendable tener un registro histórico mínimo que permita tener un cierto conocimiento del comportamiento de la estructura, y en concreto de las juntas (aperturas y cierres principalmente), que pueda apuntar a la necesidad de los trabajos de reinyección o, por el contrario, a un tratamiento más localizado que se centre en la corrección de filtraciones o humedades. Independientemente de la solución escogida, con carácter previo a cualquier actuación, convendrá investigar sobre el estado y disposición de las juntas que limitan los recintos de inyección de juntas, circunstancia que puede condicionar los trabajos de inyección propiamente dichos.

Una finalice la implantación del sistema de auscultación previsto, incluyendo el acondicionamiento de los dispositivos previamente existentes, se encontrará en condiciones de realizar un seguimiento del comportamiento de la presa de Villagatón acorde con la entidad de la estructura de cierre y de las características de su cimiento. Convendrá igualmente establecer una sistemática de redacción de informes anuales de comportamiento por parte de especialistas en la materia, al menos en los primeros años, pudiendo prorrogarse por períodos de tiempo más amplios si se considera conveniente.

La metodología y los resultados obtenidos resultarán de interés para actuaciones similares futuras.

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Presa de Susqueda. Reinyección de juntas

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ABSTRACT: Susqueda vault, 135 meters high, was built between 1963 and 1967. He has always had a comprehensive system of monitoring and has been the subject of frequent studies of its behavior. From these studies, we found it necessary to reinforce the structure re-injecting their vertical joints. The injection was performed with two component resins that had of placing and environmental advantages. It took a few years later analysis of monitoring to know with some precision the obtained results: a measurable increase in the length of the arcs injected and a comeback of the structure towards a hardened behavior, even in cold weather and with a low hydraulic load.

1 INTRODUCCIÓN

La presa bóveda de Susqueda fue terminada en 1968 y ha dispuesto de análisis de su comportamiento desde la época constructiva hasta nuestros días; éstos estudios han estado siempre basado en los datos de un sistema de auscultación extendido a, prácticamente, todos los parámetros de interés; cabe destacar la existencia de termómetros, extensómetros de hormigón, medidores de juntas eléctricos y manuales, extensómetros de varilla e hilo, piezómetros, aforadores, etc. Sobre el comportamiento de la presa se han elaborado casi tantos informes anuales como edad tiene la estructura y cuatro informes recopilatorios de carácter decenal, en general extendidos hasta la época de construcción; estos informes recopilatorios han tenido diversas autorías. También se cuenta en el archivo técnico con tres informes emitidos durante la época de construcción.

Este esfuerzo puesto en el estudio ha llevado a un buen conocimiento del comportamiento de presa y cimiento y a la necesidad de acometer algunas actuaciones en diversos períodos.

En lo que concierne a este escrito, cabe notar que el trabajo estructural de la bóveda se ha encontrado, en tiempos recientes, menos adecuado y con repercusiones que conviene corregir; estableciéndose una recomendación, coincidente en varios autores de los estudios citados, sobre la conveniencia de reinyectar las juntas.

Resumiendo mucho y simplificando, el principal aspecto del comportamiento, que hace recomendable la reinyección de juntas, reside en que, como sucede muchas veces, la inyección de juntas posterior a la construcción no pudo realizarse en condiciones idóneas, rellenándose recintos que todavía tenían calor de hidratación por disipar. Ello condujo a aperturas posteriores, cuando las temperaturas alcanzaron su proceso cíclico de equilibrio, aperturas que afectaron más a los niveles de las galerías 1, 2 y 3 que tuvieron hormigonados más tardíos.

Esta apertura, que se manifestaba más cuando el embalse estaba más bajo, ha venido mostrándose evolutiva en el tiempo produciéndose, con el paso del tiempo, cada vez para cotas de embalse más altas.

En la Figura 1 puede notarse que, durante la construcción, fue necesario dejar cuatro bloques centrales retrasados para hacer frente a posibles avenidas; su hormigonado posterior produjo un calor de hidratación que no llegó a disiparse del todo para la época de inyección de juntas.

2 PROBLEMA PLANTEADO

A partir de los estudios realizados en diversas épocas y por diversos autores se concluyó sobre la necesidad de reforzar la estructura reinyectando sus juntas verticales. Cierta falta de monolitismo llevaba a la estructura a un comportamiento inadecuado, sobre todo cuando las cargas introducidas por el embalse no eran fuertes (nivel de embalse relativamente bajo).

Se encargó un proyecto de inyección de juntas que tenía como prólogo imprescindible la realización de un nuevo estudio recopilatorio, unos diez años después del redactado en 1993,

Figura 1. La seguridad hidráulica durante la construcción forzó el retraso de puesta en obra de cuatro bloques, influyendo mucho en la situación térmica.

que debía profundizar mucho en las juntas, su comportamiento y cómo inyectarlas y en las repercusiones estructurales que cabía esperar de tal inyección.

El estudio produjo un listado de conclusiones que se escapan del contenido de esta ponencia pero pueden destacarse aquí las siguientes:

– La primera puesta en carga parcial produjo deformaciones relativamente importantes. En el año 1967 con la presa prácticamente finalizada pero sin haberse inyectado aún sus juntas de contracción, tuvo lugar una avenida que ocasionó una puesta en carga parcial de la presa importante. (Cabe resaltar aquí que no se debe plantear un plan de auscultación para explotarlo una vez terminada la presa, toda vez que durante la construcción pasan, o pueden pasar, acontecimientos de mucha mayor repercusión que los que puedan llegar en la explotación ordinaria. Por ejemplo un llenado parcial con la presa sin terminar y con las juntas sin inyectar, como pasó en Susqueda).
– La relativa ineficacia de la primera inyección de juntas que encontró parte de la presa todavía caliente, al no haberse producido completamente la disipación del calor de fraguado y registrándose enfriamientos posteriores de hasta 20ºC. Esta pronta inyección debió estar muy justificada en su momento y, probablemente, cubierta por la previsión de una posterior reinyección, si hiciese falta.
– Cuando la presa se descarga en primavera, con el arco superior ya calentando se conjugan dos fenómenos. En el primero, el arco superior intenta llevar la presa hacia arriba; la carga escasa no puede cerrar las juntas más abajo y los bloques quedan relativamente sueltos—salvo en la parte superior. Comparando aperturas de juntas en las primaveras de 1990 y 2002, en ambas con embalse bajo, respectivamente 294.00 y 304.05, pudo notarse que se habían dado cierres generalizados en el nivel de la galería 1 y aperturas, casi generalizadas, en el nivel de la galería 3; es decir, los esfuerzos de una pequeña carga hidrostática quedan concentrados en el nivel de la galería 1. Este aspecto del comportamiento tuvo que influir notablemente en el segundo fenómeno, la generación de una fina fisuración, predominantemente horizontal, que llegaba a tener continuidad desde el paramento de aguas arriba hasta el hastial de la galería, con presencia de agua, apenas perceptible, en la galería 1 y que afectaba a, prácticamente, todos los bloques.

– Por lo demás, el comportamiento de la presa, seguido de forma exhaustiva desde su puesta en servicio, se ha encontrado siempre correcto, no indicando ninguno de los elementos de auscultación instalados cambios significativos en los parámetros fundamentales de estado de la presa que pudiesen indicar la necesidad de variar las condiciones de explotación.

Tratándose, como se trataba, de realizar una reinyección de juntas, con más conocimientos ahora que los que hubo en la primera, había que buscar unas condiciones para la inyección que garantizasen un éxito importante.

Desde luego, una condición fuerte era la carga hidrostática, con las juntas cerradas difícilmente se lograría una reinyección adecuada.

Por lo dicho anteriormente, resultaba evidente que se tendría que utilizar el sistema de auscultación para ayuda en las decisiones y para analizar los resultados según se produjeran.

Se planteó la reinyección en dos campañas por el número de recintos a inyectar, para conseguir márgenes de tiempo para actuar y, sobre todo, porque las previsiones sobre cuando se podría inyectar lo exigían.

Las condiciones para la inyección pedían un nivel de explotación lo más bajo posible, condición totalmente contradictoria con las garantías de abastecimiento que se demandaban en el 2005. En este punto hubo una cierta fortuna y la importante sequía ayudó a conseguir una cota baja.

Durante las gestiones para bajar el nivel de embalse se constató claramente la dificultad de su consecución lo que ayudó a corroborar que inyectar lo más importante, estructuralmente hablando, en la primera campaña era la mejor solución: podría ser muy difícil el conseguir nuevamente otra cota baja al año siguiente.

Otros condicionantes de la inyección recomendaron la no utilización de productos cementicios (no se podía perder tiempo atendiendo a la reparación de fugas en paramentos, tampoco podían permitirse pérdidas de lechada al cauce, había que inyectar algunos recintos con aperturas de pocas décimas, etc.).

Los condicionantes empujaban hacia el empleo de resinas y faltaba contemplar el factor económico, que no fue obstáculo. Se decidió utilizar resinas ya en las primeras pruebas de inyección.

En los dos apartados siguientes se resumen las comparaciones hechas entre usar productos cementicios y usar resinas.

3 INYECCIÓN DE JUNTAS CLÁSICA Y SUS PROBLEMAS

Enfrentados con el problema de la reinyección se pusieron encima de la mesa las reglas clásicas que rigen una inyección de juntas. Se exponen aquí, también de una forma resumida:

– Si se inyecta el recinto de una única junta, se puede abrir con la presión de inyección. Las adyacentes cerrarán.
– En pequeñas presas se inyecta todo un nivel de una vez. En las grandes esto es imposible y se inyectan, a lo sumo, en grupos.
– Siempre se controlan las aperturas con comparadores de resolución una o varias milésimas de milímetro. Se recomienda colocarlos lo más cerca posible del nivel superior del recinto que se inyecta.
– Con el estándar de recintos de 15 metros de altura, las aperturas máximas no deben exceder de 0,5 mm.
– Se proponen controles con teodolito o péndulo para limitar los movimientos de los arcos hacia aguas arriba causados por las presiones de inyección.
– Se sugieren presiones de inyección 2 a 3,5 kg/cm^2 de presión. Precisando más las especificaciones se propone la fórmula *Presión en colectores* <= *Presión en inyectores* + 0,2* *altura del recinto*, limitando la *Presión en colectores* a menos de 3–5 kg/cm^2.
– Las inyecciones se realizan de manera continuada desde cimentación hacia coronación.
– La inyección se realiza cuando la temperatura de la presa es mínima, usualmente a finales de primavera.
– Se requiere un personal muy experimentado.

Al panorama anterior había que añadir otros aspectos derivados de experiencias recientes habidas en inyecciones de presas de entidad similar a Susqueda:

– A no ser que se esté por inyectar una bóveda de muy poca altura, los distintos niveles de recintos presentan su época de máxima apertura más cerca de principios de año cuanto más cercanos están a la coronación y, en una presa de cierta entidad, cuando se puede inyectar un recinto próximo a la cimentación de los bloques altos, los recintos cercanos a la coronación ya cerraron.
– Los planteamientos teóricos sobre las presiones de inyección a emplear en conductos de inyección o purga se muestran muchas veces en discordancia con la realidad y tienen que ser sustituidos por otros más realistas. Concretamente, pensar que la presión en colectores es la suma de la que hay en inyectores más la carga hidrostática del fluido inyectado es ignorar las fuertes pérdidas de carga que suele exigir la inyección de un recinto no muy abierto.
– Las presiones en los recintos adyacentes al que se va a inyectar, muchas veces colaboran a un reparto de cargas y a exigir menos a los bloques

cuya junta se inyecta; pero en otras no se necesita esa colaboración y las presiones laterales están contribuyendo a cerrar el recinto que se inyecta y a dificultar el trabajo, o impedir su realización correcta.
– La utilización de sistemas modernos de adquisición de datos automatizada y la ampliación del control de juntas a casi todos los recintos de inyección proporcionan una información muy valiosa para la toma de decisiones en los trabajos de inyección, limitando siempre los efectos negativos sobre la estructura.

O aspectos derivadas del conocimiento de la propia presa:

– Las aperturas de juntas habidas en Susqueda eran muy variadas y se producían a lo largo de un amplio intervalo de tiempo que superaba los dos trimestres. Como era de esperar, en general las máximas aperturas se van retrasando en el tiempo a medida que se baja en cota.
– Muchas de las juntas están teniendo máximas aperturas de muy pocas décimas de milímetro, difíciles de inyectar con productos cementicios.
– Las condiciones que pedía la labor de reinyección, atendiendo a la cota de embalse recomendable para la inyección, y las condiciones de explotación, a "negociar" por así decirlo, en el invierno y principios de primavera de 2005, estaban bastante encontradas.

4 SOLUCIÓN ADOPTADA EN SUSQUEDA

Con este panorama de dificultades, con un conocimiento bueno de la presa tanto respecto a las correcciones más convenientes como a sus limitaciones, y con el planteamiento claro de que no se podría realizar un trabajo perfecto pero sí uno óptimo, se diseñó una metodología de inyección con los siguientes objetivos:

– Se tendría que realizar en dos campañas para conseguir inyectar cada conjunto de recintos en la época más conveniente; tanto desde el punto de vista de que tuviesen una apertura suficiente, como desde el de afectar menos a la explotación.
– Determinados niveles de la presa necesitaban tener sus recintos muy rellenos para lograr una mayor rigidez estructural de la bóveda, frente a la carga hidrostática; en otros, ese condicionante era menor.
– El conocimiento adquirido sobre la evolución de las juntas, aun siendo bueno para determinar el comportamiento de la presa, se consideraba escaso para atender a los detalles de una reinyección de juntas; el propio control de los trabajos de inyección habría de proporcionar información para reorientar trabajos de inyección sucesivos.
– Los trabajos de inyección de juntas con cemento producen abundantes residuos, los retornos de inyección no son aprovechables y las fugas tampoco. En la época de construcción, las afecciones al medio ambiente son más tolerables y tienen, incluso, una labor posterior de recuperación. En el paraje que aloja la presa de Susqueda era preciso ser muy exigentes en impedir cualquier afección al embalse o cauce.
– La previsión de costes se había hecho pensando en inyectar lechada de cemento y microcemento. Cualquier variante debía atenerse a ese presupuesto.

Contemplando uno de los problemas, la limitada apertura que presentaban algunas juntas, se planteó una primera alternativa, el empleo de resinas como producto a inyectar. Las dificultades que podía presentar el uso de este producto se pensaba que iban a estar en el empleo de grandes presiones de inyección que podían afectar a la estructura y en el coste final al utilizar un producto mucho más caro.

Se diseñó una resina epoxi adaptada a las necesidades, fundamentalmente bajo módulo de elasticidad, fácil inyectabilidad y que no trasmitiese tracciones fácilmente, y se realizaron pruebas en dos recintos relativamente dispares.

Mientras se estudiaban las posibles alternativas de inyección, se había complementado el sistema de auscultación de la presa instalando unos 250 comparadores, un par en cada recinto, y 40 ternas de bases para elongámetro en juntas ya controladas con comparadores. A esta instalación siguió un esfuerzo notable en trabajo de adquisición de datos, con dos campañas por semana, e interpretación de los resultados que se hacía una vez a la semana.

Los resultados de las pruebas fueron muy positivos en varios aspectos:

– Los movimientos máximos registrados fueron muy limitados y claramente por debajo de los límites establecidos. Se anotó una apertura máxima de 0,30 milímetros en la junta inyectada y entre 0,02 y 0,01 mm en las adyacentes, con ausencia de movimientos en la junta inyectada, en los niveles superior e inferior al rellenado.
– El proceso se ajustaba a las necesidades muy bien, con tiempo de ejecución, incluidas labores preparatorias, acortado y gran limpieza en la ejecución.
– No se produjo ninguna fuga de resina hacia el exterior del recinto aunque se había anotado una salida de agua en las pruebas previas.
– Extrapolando el volumen inyectado a los recintos previstos se estimó un coste muy similar al previsto para un uso de lechada de cemento.

– Con estos resultados, se acometió la primera campaña, prevista para iniciarse a primeros de junio, según los resultados de los estudios previos, y que se pospuso hasta primeros de julio atendiendo al análisis continuado de situación de juntas que se venía realizando.

5 NECESIDADES DE INYECCIÓN SEGÚN RECINTOS

Se ha citado que las necesidades de inyección de los distintos recintos que conforman toda la altura de una junta eran diferentes para cada uno. Esta afirmación merece una explicación ya que determinó totalmente la planificación de las campañas de inyección.

En efecto, la bóveda venía mostrando juntas más abiertas cuando la carga de embalse bajaba mucho, lo que es lógico. También mostraba la existencia de un arco superior sin esas aperturas, ni siquiera cuando la cota de embalse bajaba mucho. Gracias a los análisis de comportamiento efectuados sobre Susqueda, se sabía que esa zona alta presentaba muestras de expansión, asociada a cierto deterioro del hormigón. Para la extensión limitada de esta comunicación hay que resumir diciendo que la presa tenía un arco superior fuerte, reforzado por leves procesos expansivos, y arcos más bajos debilitados por un estado de juntas no adecuadamente rellenas, contribuyendo ambos aspectos en la misma dirección, para un comportamiento inadecuado.

En la situación anterior, cuando el embalse estaba relativamente bajo y con el arco superior muy reforzado por el proceso comentado y porque las temperaturas calientan mucho el paramento de aguas arriba, orientado al sur, y no protegido por el agua, la presa tenía sus ménsulas relativamente sueltas, salvo en la parte superior. En el calentamiento de primavera dominaba el calentamiento del arco superior sobre cualquier otra acción llevando la presa hacia arriba y abriendo las juntas y, lo que ha sido más determinante para la fisuración aparecida, colgando la parte inferior de las ménsulas del arco superior. Desde luego no se puede pensar en un arco sosteniendo ménsulas colgadas, sino en una ménsula que se enfría y suelta de las de al lado. Esta debilidad no era nada conveniente, aunque se daba en situaciones de carga baja y con el peso propio predominando como solicitación. El refuerzo de las uniones de juntas verticales resultaba muy necesario.

Por lo dicho, no era aconsejable reforzar el arco superior alrededor de las galerías 1 y, probablemente, 2; la galería 2 se dejó para una segunda fase, planteando no inyectar alrededor de la galería 1. La galería 3 y la galería 4 eran las que necesitaban una inyección más cuidada.

6 PROCESO DE INYECCIÓN

Como se ha dicho, para evitar reforzar más el arco que rodea la galería 1, se decidió dejarla sin inyectar. La galería 2 participa algo de la situación expandida de la galería 1, por ello se dejó para una segunda campaña. Las galerías 3 y 4 están rodeadas de dos arcos que convenía reforzar mucho. Por ello y porque el proceso de inyección tenía que estar muy ligado a temperaturas frías y juntas abiertas, se definió el orden lógico, desde el punto de vista térmico: en la primera campaña, primero la galería 3 y después la galería 4 y siguientes, y había que empezar el 1 de junio y terminar, para estos niveles relativamente altos, el 31 de julio. El programa se cumplió bien, con leves retrasos admitidos por la evolución de las juntas.

Los conductos de inyección existentes resultaron inutilizables y hubo que realizar unos nuevos, consistentes en taladros practicados desde las galerías que intersectaban a los planos de los recintos en ángulos pequeños. No se aspiró a inyectar de forma uniforme todo el recinto, por el contrario, se dispusieron los nuevos conductos para obtener un área suficiente de reparto, compatible con que la resina no llegase a los waterstop. En la Figura 2 se

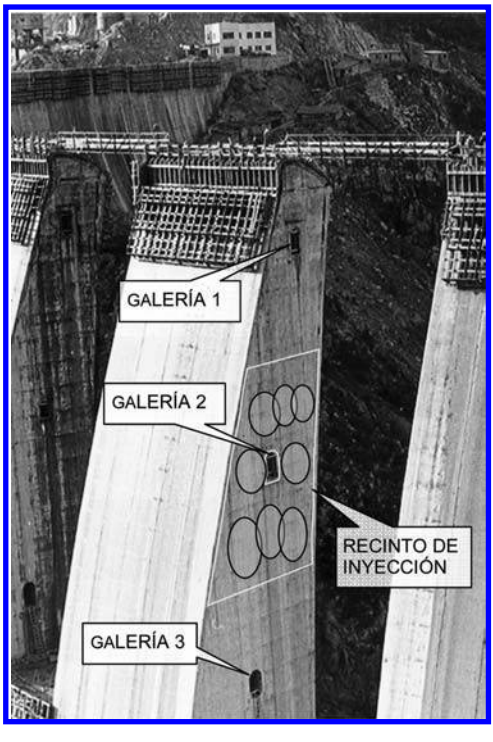

Figura 2. Áreas inyectadas con resina. La representación es teórica, la realidad resulta menos regular.

muestra el recinto y la disposición de las áreas inyectadas, aprovechando una foto antigua.

El proceso de inyección de un conducto terminaba cuando:

– La resina salía por el conducto siguiente.
– Los límites de aperturas eran alcanzados.
– Se producían fugas de resina (rara vez ocurrió).
– No había admisión.

En la segunda campaña, ya durante 2006, se inyectaron los recintos de la galería 2 y los de las galerías 6 para abajo, en este caso con menos exigencias respecto a la época del año en que debía inyectarse.

7 RESULTADOS DE LA PRIMERA CAMPAÑA

La primera campaña se realizó durante el verano de 2005. El bajo nivel del embalse en esta época, algo fomentado para realizar el trabajo, fue muy favorable para la obtención de resultados (ver Figura 3). En la Figura 4 se muestran los efectos de la inyección, vistos cuando se estaba finalizando esta campaña. En el eje horizontal se han definido intervalos de dos semanas, muy útiles para la interpretación de entonces, y, en el eje vertical, van las aperturas medias por arco, expresadas en centésimas de mm.

Las conclusiones más importantes, ya conocidas al final de esta primera campaña, fueron los siguientes:

– Las juntas de la galería 3 se inyectaron en su momento de máxima apertura, algo más tarde que lo que se esperaba por el historial analizado, y se amplió su apertura en el proceso de inyección. Tras la inyección hay algún cierre,

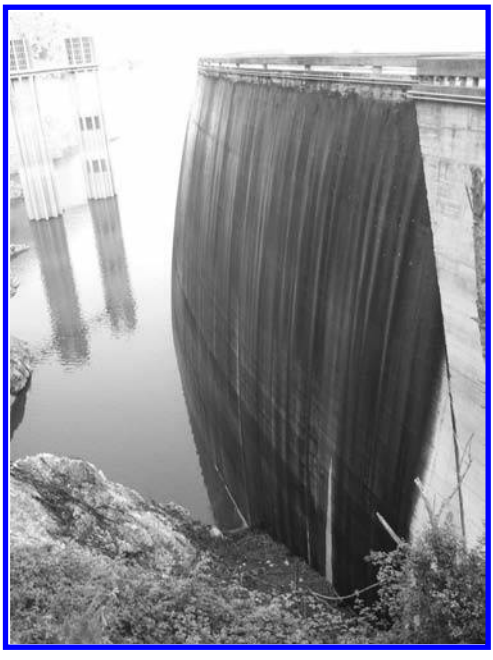

Figura 3. Importante vaciado del año 2005.

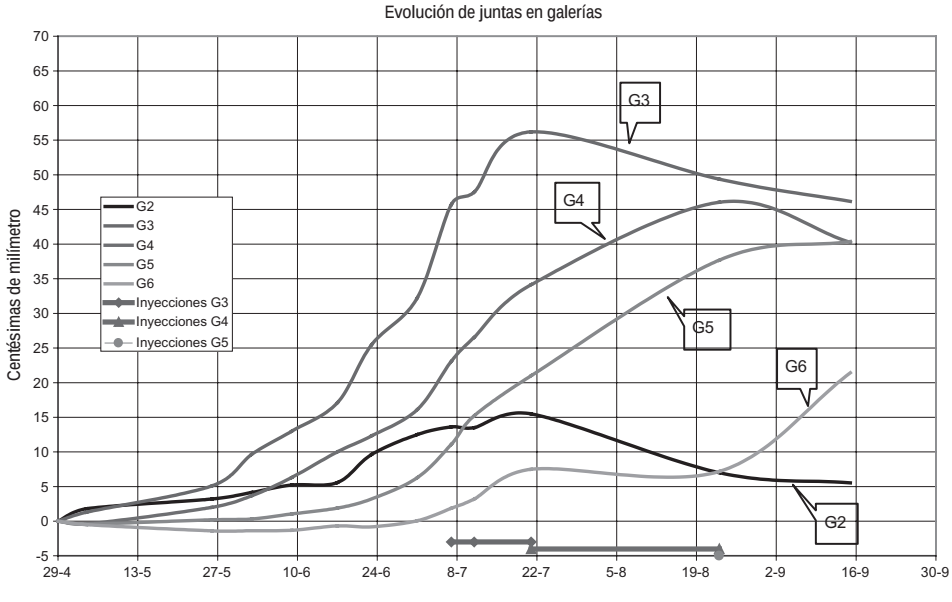

Figura 4. Evolución de las juntas durante el proceso previo y de reinyección en la primera fase.

esperado, cuya explicación puede tener varios orígenes: los recintos extremos de este nivel estaban todavía sin inyectar y cerrando; la resina estaba comprimiendo sometida a las fuerzas que ejercen los bloques, que dilataban por calentamiento; hubo recintos que quedaron menos inyectados y cerraban más, etc.
- Las juntas en la galería 4 y en la galería 5 se comportaron en el reinyección de manera similar a como lo hicieron las de la galería 3.
- En la galería 2, todavía sin inyectar en la primera campaña, se produjeron aperturas originadas por las inyecciones de recintos inferiores, registrándose máximos retrasados en el tiempo artificialmente respecto a su época anual normal.
- El incremento de apertura de la galería 6, procesado al final del gráfico, es debido a los trabajos de inyección, el arco todavía no estaba inyectado.
- Las máximas aperturas en las galerías 3, 4 y 5 se sucedieron en el tiempo, empezando de arriba, donde es mayor la exposición al calentamiento, hacia abajo, donde la presa está más protegida térmicamente; puede notarse que ello está en concordancia con el proceso de inyección previsto, que siguió esa sucesión de arriba abajo y que resultó acertado.
- Las inyecciones hechas en arcos superiores retardan en el tiempo la aparición de máximas aperturas en arcos inferiores, con lo que se pudo prolongar así, artificialmente, la época idónea para la realización de las inyecciones.
- Las máximas aperturas producidas en el proceso de inyección de cada recinto se limitaron siempre a 0,50 milímetros, como estuvo especificado en un principio.
- Los cierres producidos en juntas adyacentes a las inyectadas fueron casi despreciables, contabilizándose máximos de unas milésimas de milímetro. Los cierres producidos en segundas juntas, contadas a partir de la que se estaba inyectado, fueron inapreciables.

La existencia de dos comparadores controlando la evolución de cada recinto, en cada hastial de la galería, permitió observar que las juntas reaccionaban abriendo la zona que recibía presión de resina y manteniendo, o cerrando algo, la zona del recinto inyectada previamente.

8 RESULTADO FINAL

Al final del año 2007 se pudieron comprobar resultados estructurales de la inyección realizada al bajar el embalse a cotas próximas a la 305. Las principales conclusiones, recogidas en el informe anual, se pueden resumir así:

- Del análisis histórico de las deformadas obtenidas de los péndulos se deduce, para fechas tan lejanas como 1969, un funcionamiento homogéneo y compensado de la bóveda por encima de la galería 3, con el transcurso de los años, ese funcionamiento ha ido desapareciendo. La zona superior de la bóveda, debido sobre todo a la precaria situación en se han hallado sus juntas, se ha ido gradualmente sobrecargando con el transcurrir del tiempo. El trabajo estructural de la bóveda para la combinación de bajas temperaturas y altas cargas hidrostática ha sido bastante precario. Todo ello condujo a la necesidad de la reinyección.
- Dada la evolución de las aperturas de las juntas durante el proceso de reinyección, es muy probable que se haya incrementado la rigidez de una franja en la bóveda, centrada en el entorno de la galería 3. Lógicamente, el futuro comportamiento de la bóveda, en especial con bajas temperaturas y altos niveles de embalse, tenderá a favorecer el trabajo en arco en la referida zona de la bóveda.
- Tras la reinyección de las juntas, en la parte alta de la bóveda, en la zona comprendida entre las galerías 3 y 2, se observa una cierta reacción tendente a disminuir, para situaciones de carga similares, los desplazamientos hacia aguas abajo. Todo apunta a que la reinyección de los recintos desde las galerías 3 y 2 ha tenido su repercusión en el sentido de incrementar la rigidez de la parte de la bóveda situada a esas cotas. Dicho efecto parece desvanecerse en el nivel correspondiente a la galería 1 (la planificación de la reinyección se hizo para lograr esto).
- Con la excepción de las juntas de la galería 1 (sus recintos no fueron reinyectados), con generalidad, las juntas auscultadas alcanzaron su máxima apertura histórica inmediatamente después de la reinyección. Acumulando aperturas por arcos, se consiguió una apertura acumulada de 9.2 mm en G-3, 6.9 mm en G-4, 5.7 mm en G-5, 6.3 mm en G-6 y 3.1 mm en G-7.

En la Figura 5 se muestra la evolución de cuatro juntas medidas en la galería 3, que son bastante representativas de lo que sucede a este nivel.

Pueden notarse las aperturas producidas en los desembalses importantes de 1989 y 1990, y una repetición del proceso en los desembalses desde 1999 hasta 2002.

También está claro que los niveles de embalse altos, por encima de la cota 330 cierran bien las juntas durante todo el año.

La reinyección de 2005se nota claramente. Dos de las juntas, la 15–17 y la 19–21 quedaron bien rellenas.

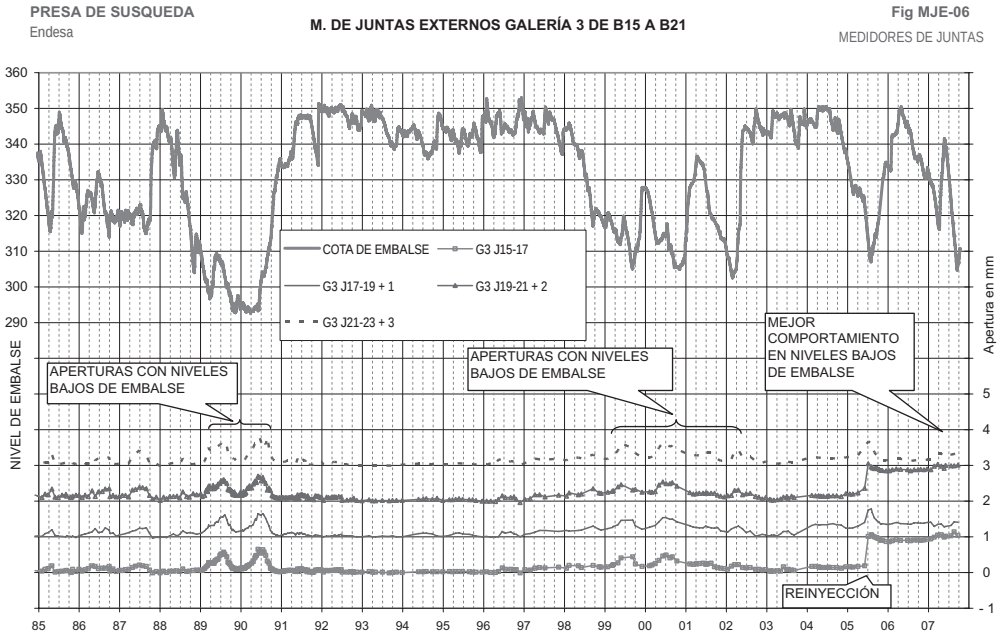

Figura 5. Evolución, anterior y posterior a la reinyección, de cuatro juntas de la galería 3.

La segunda campaña de reinyección, realizada durante 2006, no tiene influencia en los recintos ya inyectados.

Durante los niveles bajos de embalse de 2007, no se producen, o se producen en mucho menor grado, las aperturas que había antes de la reinyección.

9 CONCLUSIONES

La reinyección de juntas que se ha realizado en la presa de Susqueda se ha separado mucho del modo tradicional de inyectar las juntas. El cambio vino obligado por necesidades técnicas estructurales (no se podía sobrecargar los cierres de los recintos y se debía precisar mucho dónde se inyectaba) y medio-ambientales (no se admitió la posibilidad de tener un vertido de lechada hacia el embalse). El cambio ha significado un paso más en un proceso de mejora del estado del arte, donde, sin embargo, se han aprovechado enseñanzas y estudios anteriores sobre inyecciones más clásicas.

En este pequeño salto técnico, cobra un especial protagonismo el empleo de resinas como nuevo material de relleno que puede ser muy flexible cuando se le aplican cargas, se puede colocar con bastante precisión, permite controlar, también con precisión, las deformaciones producidas y del que cabe esperar una durabilidad amplia.

También hay que destacar la parte aportada por los estudios previos realizados sobre el comportamiento de la presa, estudios que han permitido determinar qué recintos había que inyectar, en qué fechas hacerlo y cuales otros no; También ha sido notable el seguimiento exhaustivo de los movimientos de las juntas, antes, durante y después de ser inyectadas.

La bóveda ha quedado reforzada en cotas medias y altas, allí donde se mostraba más débil, manteniendo la situación de refuerzo artificial, no deseado, en cotas por encima de la galería 2.

NOTA FINAL

Los trabajos de reinyección fueron realizados por HCC (Hidráulica, Construcción y Conservación) cuyos técnicos desbordaron de forma continuada sus cometidos típicos, con propuestas de mejoras en el empleo de técnicas y materiales.

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Obras de consolidación del dique de El Bosque de Béjar (Salamanca)

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RESUMEN: Las obras de consolidación de un dique histórico han posibilitado, en el proceso de su ejecución, el hallazgo y documentación de diferentes elementos constructivos precedentes, mayoritariamente murarios. Se trataría de una presa refundada seguramente en la segunda mitad del siglo XVI sobre otras estructuras anteriores, asociadas al lecho de una vaguada natural y a sus condiciones hidrológicas. Las actuaciones contemplaban dotar a este dique de todas las condiciones de seguridad y estabilidad que requiere un embalse en explotación, dentro de una restauración por fases a la que se está sometiendo la Villa Renacentista de Recreo de El Bosque, en la que se ubica. El dique se puede relacionar con otros que ejecuta la corona española en época filipina, en Ontígola (Aranjuez), en La Casa de Campo de Madrid, en la Fresneda (San Lorenzo de El Escorial) y en otros sitios reales.

1 ANTECEDENTES—INTRODUCCIÓN

La restauración que se presenta corresponde a la del denominado dique de la Villa Renacentista de El Bosque. Se ubica en la zona central de la finca, siendo esta de modelo renacentista italiano, dispuesta en terrazas con un estanque central de unos 4.322 m^2 en planta, con unas terrazas inferiores a poniente, con un Jardín Histórico una y otra destinada a huerta frutícola, circundado este núcleo central aparecen praderías y un monte de castaños a Sur, formando un conjunto unitario cercado por una valla de piedra, con más de 33 ha de superficie en proyección horizontal.

Las características de El Bosque han quedado expuestas en un trabajo anterior[i], con cierto detalle, su historia, paralelismos, aspectos compositivos, descripción de los diferentes ámbitos y elementos, como el sistema hidráulico y especialmente del dique y el estanque central.

La intervención acometida en las obras promovidas por la Consejería de Cultura y Turismo de la Junta de Castilla y León tienen por objetivo, entre otros, el de estabilizar y dotar de condiciones de seguridad y solidez suficiente a la presa que remansa el agua del estanque por el lado del oeste, en la situación actual de explotación efectiva para uso lúdico ornamental, como lámina de espejo y para el riego de arriates[i] y terrazas inferiores, con jardines y huertas. Se trata de la tercera fase, de las que ha promovido la Comunidad, encaminadas a la recuperación integral de la Villa, que cuenta con declaración de BIC, en la categoría de Jardín Histórico.

En el sentido reseñado por la jardinera paisajista Consuelo Martínez Correcher es la única villa en España, de fundación renacentista, que conserva intactos su trazados y los diferentes ámbitos y espacios originales, perfectamente relacionados e interdependientes, espacial, compositiva y medio ambientalmente.

El sistema hidráulico, que se recuperó en una obra inicial, siguiendo las directrices del Plan Director, como el resto de las actuaciones, de forma lógica y coordinada, es complejo y elaborado.[ii] Consta de arcas, estanques, manantiales, caceras, canales en caja cerrada, regaderas en tierra y diversas fuentes. Se consolidó y es operativo en casi su totalidad. Se documentaron atanores con enchufe y campana, de piedra granítica en posiciones originales, así como encañados con arcaduces, no operativas, y otros conductos de plomo, correspondientes a sucesivas

Figura 1. Fotografía aérea de la Villa y su entorno. Años 60.

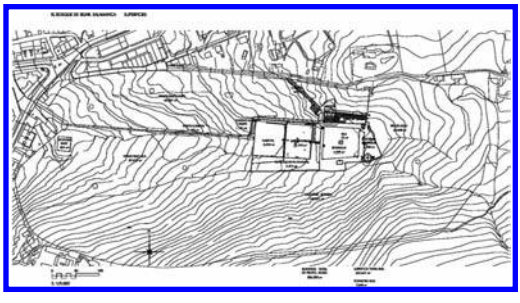


Figura 2. Plano general y superficies.

fases y periodos históricos, de gran interés. Uno de sus elementos más significativos y relevantes es el conjunto del estanque central y su dique, junto con otro estanque de menores dimensiones, situado en el extremo occidental de la finca, en la parte baja a menor cota, denominado Estanque del Tinte. Este servía a una de las primeras industrias conocidas en España, coetánea a la Real Fábrica de la Moneda en Segovia, hacia 1590, un tinte, y antes a un batán. Los restos del Tinte del Duque desaparecieron en 2001, entre otras causas, por la presión inmobiliaria.

En una fase intermedia se restauraron tres fuentes históricas, dos de la segunda mitad del siglo XVI y otra posterior barroca.

Se trata de un dique que en su construcción y tipología, y en parte, sigue el modelo holandés aplicado en otras obras contemporáneas del reinado de Felipe II.

Inicialmente se acometió un estudio exhaustivo de este tipo de presas, principalmente documental, tanto de las existentes en Ontígola y la Fresneda, como las desaparecidas en la Casa de Campo de Madrid y las probables de Valsaín, en Segovia.[iii]

Fueron de gran ayuda los diferentes estudios históricos específicos realizados por el Grupo Cultural San Gil, de Béjar, concretamente por los estudiosos, pioneros en El Bosque, José Muñoz Domínguez, Juan Félix Sánchez Sancho y Urbano Domínguez Garrido entre otros, recogidos en las Actas de las Jornadas sobre *"El Bosque de Béjar y las Villas de Recreo en el Renacimiento"*.

Se realizaron varios estudios geotécnicos, en dos campañas, por una empresa especializada, Inzamac, dejando en la segunda una línea de piezómetros a lo largo del dique, con varios metros de profundidad, que se han ido siguiendo hasta el inicio de los trabajos sobre el dique, y diversas catas arqueológicas superficiales en el núcleo terrero de la presa y en su entorno.

Todo ello ha servido para realizar una evaluación de su estado y elaborar los cálculos de sus condiciones estáticas, para definir la estrategia de intervención, dentro de un equipo pluridisciplinar, con la que se definió el contenido del proyecto de ejecución.

Este proyecto contempla también una recuperación, en la línea de la restauración de la jardinería histórica, de la plantación y tratamiento vegetal de las cuatro calles del estanque.

El sub-proyecto de jardinería fue elaborado conjuntamente con ETF, Empresa de Estudios y Trabajos Forestales y la valiosa colaboración de José Muñoz.

Seguidamente se realizó un diagnóstico completo que sirvió para plantear una propuesta de rehabilitación que contempló aspectos de limpieza, consolidación, restauración y otros de estabilización que determinaron la conveniencia y necesidad de una refundación parcial de parte de la estructura del conjunto, con un detallado seguimiento topográfico, geofísico, y de movimientos, recogido en partidas del proyecto. Estas tareas las desarrolla

Figura 3. Presa de El Mar de Ontígola. 2007.

Figura 4. Embalse de Granjilla I, en la Fresneda. 2006.

Perímetro cerca exterior	2.645 m
Superficie total del Bosque	357.641m^2
Altitud sobre el nivel del mar	
• Zona lado sureste	1.046 m
• Zona lado oeste	952 m
Diferencia de cotas entre el nivel más alto y más bajo	94 m
Cotas s.n.m. Terrazas del estanque.	
Cota Calle norte. Extremo O 1.003,90. Extremo E 1.004,20	
Cota Calle Este. Centro terraza 1.002,70	
Cota Calle Sur. Centro terraza 1.002,00	
Cota Calle Oeste. Centro terraza 1.001,50	
Pretil norte	75 m
Pretil este	58 m
Pretil sur	77 m
Pretil oeste	63 m
Cotas lámina del agua s.n.m.	
Cota de entrada del agua en la finca	1.034 m
Cota de entrada del agua en la finca	1.026 m
Cota de salida del agua de la finca	954 m
Diferencia de desnivel del agua en su recorrido por la finca	80 m
Cota del nivel medio (lleno) del agua del Estanque	1.001 m
Superficie lámina de agua Estanque Principal	4.322 m^2
Aforo inicial estimado del Estanque Principal (antes de 1662)	21.133 m^3
Relleno de terraplenado en el año 1662	4147 m^3
Aforo del Estanque tras el terraplenado (estimativo)	16.986 m^3
Lodos y terrenos de arrastre (tarquines) depositados (0,60cm de media)	2.593 m^3

Figura 5. El Bosque en cifras.

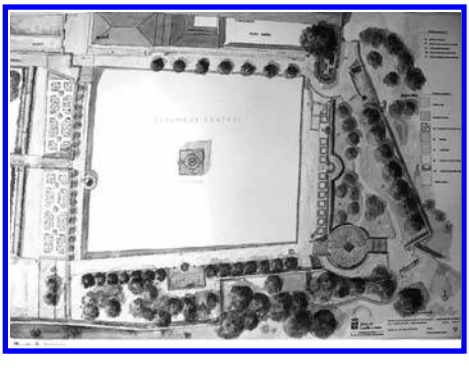


Figura 6. Plano general de imagen e intervención.

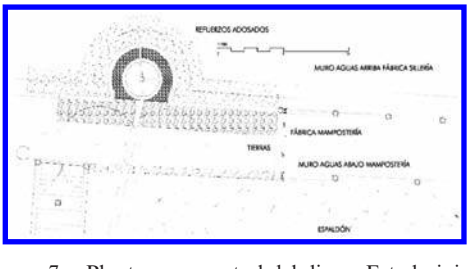


Figura 7. Planta zona central del dique. Estado inicial con arbolado.

la propia empresa que ejecuta las obras y el equipo arqueológico que atiende sus cometidos específicos, en este caso continuos y prolijos.[iv]

2 DESCRIPCIÓN ESTADO INICIAL

La presa se compone de un cuerpo longitudinal de trazado Norte-Sur, con relleno de tierra y muros pétreos o terraplén en sus laterales, una escalera central situada perpendicularmente en el costado oeste, de acceso a las terrazas inferiores, un cubo adosado por el frente de aguas arriba, denominado Cubo del Desaguadero, de planta ultrasemicircular y un cuerpo macizo de fábrica, con hoja exterior de sillares, núcleo, seguramente de cal y canto, y remate superior de coronación de bolos de cuarcita, en el rincón interior suroeste del estanque.

Esta construcción cuenta con una longitud en el muro de aguas arriba, coincidente con el lado oeste del estanque de 63 m, (por el interior del vaso) mientras que el de aguas abajo (en dos tramos de diferente longitud y constitución) desarrolla 88 m, ya que se ha de prolongar para contener la terraza del estanque por ambos extremos, al Norte enlazando con la calle superior, con una escalera, y a Sur con la plataforma de la terraza contigua.

La anchura media es de 5,5 m. y su altura oscila de los 5,5 m hasta los 11 m desde la cota se suelo acabado de la calle al nivel de cimentación en roca.

El cuerpo de la presa se constituía por un núcleo de tierras, un tipo jabres de la zona, denominado arcillón en la comarca, con algunos bolos sueltos, sin compactar y no extendidas por tongadas. Quedó sobradamente documentado, en varios sondeos, con extracción de columnas litológicas. Se constató con estas pruebas la profundidad y el nivel de firme granítico de la vaguada, en V muy abierta, entre los 5 y los 11 de profundidad respecto del nivel de la calle. Sí aparecía en el nivel superior del granito un diferente grado de alteración, y una capa superior de granito muy alterado, o jabre, de 1 a 2 m de potencia. El muro pantalla que forma

Figura 8. Fotografía área zona central con estanque y dique (a su izquierda).

Figura 9. Vista general del estanque, con el muro de aguas arriba a la izquierda, con el Cubo del Desaguadero.

Figura 10. Vista general del muro de aguas arriba del dique.

en su conjunto el lienzo de aguas arriba se constituye por una fábrica de sillería, de unos 40 cm de grosor, con hiladas horizontales y un muro adosado, en trasdosado posterior, de mampostería, de 1,70 m de anchura, salvo en un tramo de 15 m desde el extremo norte, que cuenta con 65 cm de grosor, y en este caso no constante.

La pantalla de aguas arribas presenta un asiento histórico significativo hacia el centro del conjunto, que se ha ido corrigiendo con la adición de hiladas de sillares que persiguen la horizontalidad del remate y del pretil. Los pretiles, en ambos laterales se resuelven con plintos prismáticos de aristas redondeadas, y cuerpo inferior a modo de basamento de planta cuadrada y remate superior con molduras, a manera de basa, con un orificio central en su coronación (para recibir un vástago prismático con plomo, para anclar seguramente un jarrón floral, ramilletero o paila,) y tramos de mampostería entre ellos rematados a lomo de perro.

Aguas abajo, en la mitad norte se contaba con un muro de mampostería, con doble talud, por el intradós y por el visto, con una anchura que iba desde los 80 cm en la coronación hasta los 120 cm en su arranque. En el tramo sur aparecía un espaldón de tierra, con una pendiente desde los 30 a los 40º, que en tramos sufría uno o varios quiebros, con un acabado visto de mampostería careada de tamaño medio.

Contaríamos con una presa de gravedad, constituida en gran parte por terrenos sueltos. El tramo norte correspondería a una presa tipo holandés española y en el sur a una presa de pantalla con terraplén o espaldón. La filiación e influencia flamenca aún no se ha podido corroborar de forma documental, a pesar de los esfuerzos y recursos empleados.

Figura 11. Detalle tramo sur y Cubo con muretes en seco para filtrado, previos a los desagües.

Figura 12. Muro de aguas abajo, tramo norte, (derecha) con esquina mirador y terrazas intermedias (en el frente). Estado inicial.

Figura 13. Vista lateral de la escalera central. Estado inicial.

La descripción más gráfica en términos coloquiales, y afortunadamente no del todo cierta, la aporta un geólogo que opinaba que el dique se asemeja a “un flan, alargado, con galletas pegadas por ambos lados”

Los paralelismos de las secciones de este dique con la de una presa descrita en un pliego de

condiciones histórico, son apreciables, (la reseña es del artículo "*La casa de Campo de Madrid como Complejo Hidráulicos"*, de Carmen Díaz-Guerra, y otros, editado en el Tomo I del I Congreso Nacional de Historia de las Presas) redactado por el diquero holandés Pietre o Petri o Peeter Janson, para un dique de la Casa de Campo.

Igualmente son conocidas las intervenciones de este personaje en la finca de La Fresneda, especialista con otro, en diques y embalses, y seguramente y sobre todo en piscicultura, expuestas en la publicación de Luís Cervera Vera, *"La Fresneda. Un lugar de Felipe II en el entorno de El Escorial"*.

Los contactos de la Corona con la casa ducal son conocidos, pero no he llegado a establecer una posible relación directa y documentada sobre su influencia en los aspectos hidráulicos de El Bosque, ni tampoco hemos llegado aún a conocer la mano, o los autores de la traza de este dique, con similitudes en algunos aspectos, como en el Cubo, con estanques de alguna villa italiana de la época.

Tras los cálculos y comprobaciones numéricas, y bajo diferentes hipótesis, se confirma y atestigua que el estado de estabilidad general era precario, con la constancia de filtraciones por su base, y otras, también conocidas, como son las de la pantalla del paramento de suso.

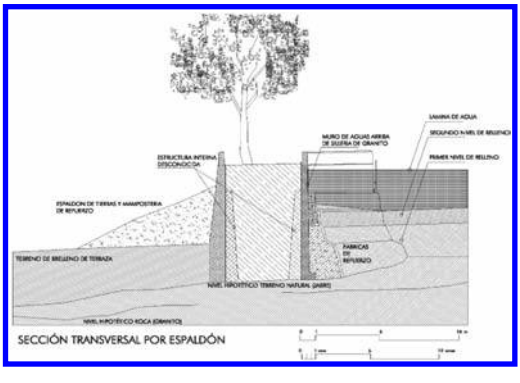


Figura 14. Esquema de una sección transversal tipo de la propuesta.

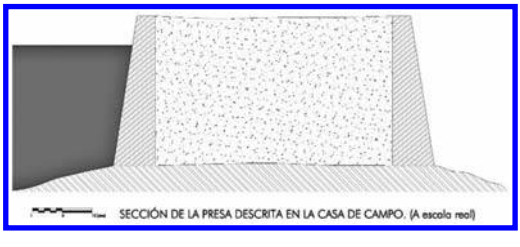


Figura 15. Interpretación de la sección de un dique para la Casa de Campo de Madrid.

FECHA	NOMBRE	RÍO	ALTURA (m)
1500 (?)	Castellar	Rivera de Alconera (Guadiana)	19,0
1530	El Embocador	Tajo	4,2
1530	Valdajos	Tajo	5,0
1565-1572	Ontígola	Ontígola (Tajo)	7,0
1566-68	Granjilla 1 y 2	Aulencia (Tajo)	13,5 y 5,5 respect.
1567(?) -1677	El Bosque	Regadera desde el Río Barquillo	Media 6 (Min 5, Max 7 -10)
1577	Albuera de Trujillo	Albuera (Tajo)	15,0
1586 (?)	Almansa	Belén-Grande (Júcar)	23,0
1594	Tibi	Monegre (Júcar)	46,0
1640	Elche	Vinalopó (Júcar)	23,0
1648	Puentes I	Guadalentín (Segura)	22,5
1693	Albuera-Casabaya	Casablanca (Guadiana)	16,8
1704	Arguís	Isuela (Ebro)	23,0
1728	Mezalocha	Huerva (Ebro)	45,0

Figura 16. Relación de algunas presas españolas coetáneas.

Los problemas, o más exactamente los condicionantes de las filtraciones serán en este caso, como en la mayoría de presas afines, de especial relevancia.

Los informes geotécnicos inciden en la existencia de filtraciones en el lecho del vaso. Lógicamente el sustrato de roca no es ni mucho menos continuo e impermeable. Se cuenta con diversas fallas de diversa entidad, fisuras y roturas. A pesar del terraplenado de 1662, que rellena casi a la mitad de su altura el vaso, con tierras más o menos arcillosas, parte del agua embalsada, tras empapar este relleno terrero, se pierde por su fondo. Las terrazas perimetrales, que se construyen en el valle inicial, junto con el dique, cada vez más altas, suponemos que por recrecimientos sucesivos, y por tanto con mayores aportes de tierras, se separan del estanque con muros de contención mampostería, los cerramientos del vaso, de escaso espesor y de obra en seco o con poca argamasa, o quizá disgregada por efecto del lavado del agua.

Por tanto se trata de un sistema poroso donde parte del caudal embalsado satura las terrazas perimetrales, y dónde parte del agua de lluvia se filtraría y pasaría al estanque a través de ellas, en un sistema fluctuante de compensación o mejor de intercambio variable. En épocas o estaciones menos lluviosas, las terrazas se saturarían con más facilidad, como esponjas, sustrayendo más agua del estanque.

En fase de obra se ha apreciado la existencia también de manantiales en la base del dique y/o en sus proximidades. Son parte de los manaderos naturales o surgencias de la finca, algunos captados en arcas para suministro y para alimentar diversas

fuentes, que aparecen por doquier, dos en el Prado Alto, uno en el Monte o Bosque, uno en la terraza lateral sur del Jardín Histórico, y otro en el Prado Bajo, al suroeste, cerca de un lugar conocido como la Mesa de la Reina.

Este es un factor que se ha tenido en cuenta en la intervención, si bien se desconoce la influencia en los manaderos de las escorrentías y filtraciones de toda la cuenca tributaria sobre el tramo final de la vaguada dónde se eleva el dique y se embalsa una porción del caudal del agua proveniente de la Sierra de Candelario Béjar, aunque indudablemente han de tener relación.

Otros aspectos respecto de las filtraciones son los que se representan en el propio asiento del dique y la pantalla de aguas arriba.

El asiento del doble muro o fábrica de dos hojas adosadas o en trasdós una de la otra, o la otra de la una (por determinar) se realiza casi siempre sobre roca, estando ésta con un grado desigual de alteración. Esta circunstancia explica la existencia de las filtraciones, que son de carácter histórico.

Tampoco la pantalla, con doble muro, es perfectamente estanca, dada la propia porosidad de la fábrica de granito, sus juntas (calafateadas ya en el s. XVII) y la de mampostería, ejecutada ésta con cal y canto de diferente factura y calidad.

Todo lo expuesto anteriormente explica la saturación de los terrenos del núcleo y el espaldón, que llegó a ser muy elevada y a poca cota de la calle (a unos 2 m en algún piezómetro).

El aporte sustancial de caudal se realiza por la denominada Regadera, cacera en tierra con algunos tramos con muretes y refuerzos de mampostería, de más de 3 Km de desarrollo que hace la toma por una derivación en canal abierto en una poza, en el Río Barquillo, de régimen estacional éste y con corto recorrido, en el tramo denominado Garganta del Oso, en el término municipal de Candelario, en la Cuenca Hidrográfica del Tajo. La concesión actual es de 120 l/s, con una servidumbre histórica, a unos prados, los Prados Domingos, que posiblemente ya desde el s. XIX disfrutan del cincuenta por ciento del caudal entre San Juan y San Miguel.

En las conocidas, por aquellos pagos, “distracciones”, no entraremos.

Un aspecto a considerar es el aporte nutrido y a veces excesivo de aguas por las vertientes de las laderas de la propia Regadera, que en época de lluvias copiosas y deshielos, y estando la toma o acometida cerrada, con piedras y sacos terreros, (ese es el sistema habitual utilizado, a mano y azada), la Regadera llega a El Bosque casi desbocada, como hemos podido sufrir en marzo pasado, con la constancia de su arruinamiento por un crecida el día de Santa Lu-cía en 1626.

La Regadera en algún invierno también se puede helar, convirtiéndose en un gran carámbano

Figura 17. Espaldón, tramo sur de aguas abajo, desbrozado.

Figura 18. Espaldón, con la vegetación rebrotada.

que impide que baje al agua a la Villa, por taponamiento.

En periodos invernales no muy lejanos, la lámina del estanque deviene en una placa sólida de hielo, parcial o totalmente, que llega a producir estallidos bastante aparatosos por compresión, al expandirse en su solidificación.

Otro capítulo especial es el papel de la acción de la vegetación invasora en estas fábricas. Béjar se caracteriza por contar con una climatología que hace especialmente propicio el desarrollo rápido, casi vertiginoso de diferentes especies vegetales, desde briofitos, plantas vasculares, a arbustos que, en algunas zonas y en pocos años pasan a ser verdaderos árboles, particularmente los alisos, especie hidrófila, aquí por otra parte especialmente protegida. Este fenómeno fue crucial en el espaldón terrero, que a pesar del forro de piedra careada se ha visto poblado de diferentes arbustos y árboles, de considerable porte, que paulatinamente, y a pesar de haberse talado en diferentes campañas de desbroce y control de la vegetación, han actuado muy negativamente en él; en concreto los sistemas radiculares que han penetrado en el relleno de tierras del talud, disgregándole, y separando y levantando el pedraplén en unas zonas y en otras casi haciéndolo desaparecer. Fenómeno habitual en presas de tipología similar.

Figura 19. Calle oeste, del dique, con arbolado de mediano porte.

Con ello se facilita la entrada de agua de lluvia, y luego hielo, las escorrentías y erosiones del propio espaldón, que se había llegado a convertir prácticamente en un motón de tierra con escasas cualidades estructurales.

En la fase de inicio de redacción del Plan Director, año 2000, se desarrollaban árboles incluso de mayor envergadura de los que aparecen reflejados, siendo estos rebrotes de los tocones no eliminados.

De forma inexplicable en la calle oeste del estanque, el paseadero del Dique, se había trazado una plantación alineada con árboles, ya podada a primeros de los años ochenta del siglo pasado, plátanos de sombra y castaños, que llegaron a un desarrollo importantes, con porte y tocones desmesurados. Estos individuos arbóreos prosperaron sobre el núcleo terrero, produciendo también sus raíces empujes especialmente acusados en el muro de aguas abajo en su tramo norte (de una hoja de mampostería). Lógicamente se talaron y destoconaron, no sin gran esfuerzo, dado que algún tocón alcanzó en sus medidas externas los $2 \times 2{,}5$ m.

Al respecto es interesante un artículo aparecido en el nº 122 de la revista Ingeniería Civil, año 2001, a cargo del Servicio de Publicaciones del Centro de Estudios y Experimentación de Obras Públicas. Ministerio de Obras Públicas, CEDES, Madrid.

3 INTERVENCIÓN

Para encarar las soluciones de proyecto fueron especialmente relevantes los artículos *"Presas y Embalses. Planificación Hidráulica y Ordenación del Territorio*, de Miguel Arenillas Parra, y *"Presas y Dehesas"*, de José Luis Mosquera Müller, en los Congresos Nacionales de Historia de las Presas.

También los eruditos estudios sobre Ontígola, en concreto los de Arenillas Parra y Segura Graiño, y otros estudios suyos sobre obras de ingeniería hidráulica.

Se tomaron como documentos de referencia *"Reflexiones acerca de la necesidad y criterios de intervención en presas históricas"* de Francisco Bueno Hernández y conjuntamente con Joaquín Díez-Cascón Sagrado las publicaciones *"Ingeniería de Presas. Presas de Fábrica"* y *"Las presas y embalses en España. Historia de una necesidad"*. Tampoco pasaron sin ser consultados libros como *"Historia de las presas"* de Nicholas J. Schnitter o manuales técnicos como *"Diseño y construcción de pequeños embalses. IRYDA. Ministerio de Agricultura, pesca y Alimentación"*.

Las filtraciones referidas se debían, a nuestro criterio mantenerse, por varias cuestiones, al menos en parte y controlando su intensidad. No se pretendía conseguir una estanquidad completa, con un gunitando por ejemplo todo el vaso, para convertir el estanque en una piscina. El carácter de bien patrimonial de máximo valor, y su rareza, por cuanto ha llegado a nuestros días casi intacto, hacen no aconsejables soluciones contemporáneas, que además de ser irreversibles no garantizaban la inspección, el seguimiento, ni el mantenimiento o reparación y restauración de los elementos ocultos por ellas, en concreto el propio vaso y los cerramientos de contención de mampuestos. Se mantenían así los criterios de restauración y consolidación empleados en otros monumentos, haciendo una traslación a este caso concreto, en la línea más consolidada y fiable de las intervenciones en el patrimonio, con la observancia del mayor respeto por el monumento, y del rigor histórico y constructivo en la propuesta de actuación y en su ejecución.

Por lo tanto se había entendido, tras consultar con especialistas jardineros y botánicos, que la red de flujos juega un papel sustancial en el sistema de riego natural de las terrazas inferiores. No nos podíamos olvidar de la terraza inferior intermedia ni de la terraza del Jardín Histórico, de unos 4.500 m^2, con coníferas, como las secuoyas gigantes (plantadas hacia 1860) y singulares calocedros (*Calocedrus decurrens*), en un conjunto jardinero romántico, correspondiente al último jardín con voluntad de estilo, del s. XIX, que se superpuso a los restos de los anteriores, neoclásico, barroco y renacentista, con un valor botánico e histórico altísimo. Algún individuo se valoraba con la Norma Granada en el año 2000 en más de 40.000.000 de las afables pesetas, por no hablar de su valor como paisaje cultural. Alterar el nivel de humedad y saturación de la terraza de estos ejemplares, cortando por ejemplo, las filtraciones de golpe, supondría ponerlos en riesgo, por el fenómeno de estrés hídrico. Por lo tanto se dispuso de un sistema de piezómetros con seguimiento continuado que aseguró que el sistema

radicular mantuviera una humedad casi constante y uniforme. Seguimiento que se mantendrá después de concluidas las obras.

Aspectos botánicos y medio ambientales intervenían en la actuación sobre el dique, y no podían ser soslayados, al contrario.

Tras el estudio de varias alternativas y los proyectos ideados para otras presas históricas, incluidas algunas de época romana, se llegó a una solución que en parte es afín, salvando las distancias, de todo tipo, a la adoptada en Ontígola, por Juan de Herrera y también a una de las propuestas, no ejecutada, para la presa de Proserpina, (con una estructura muraría con galerías de registro adosada al muro de aguas arriba). Simplificando.

Se desestimaron soluciones de pantallas de hormigón, por ser opciones agresivas, con repercusiones difícilmente cuantificables a priori, y además estaban las cuestiones de las filtraciones y la permeabilidad. Tampoco se consideró adecuada la solución de una pantalla de pilotes, contra el muro de aguas arriba, intermedia o contra el paramento de yuso, (luego se vería que éste solo existía en la mitad norte) separaros a intervalos, que contuvieran el empuje de la presa o la estabilizaran, pero no que cortaran el sistema de flujos y filtraciones. El inconveniente de esta solución, como otras similares, y en ocasiones en el de inyecciones, es que se desconocía la naturaleza interna real del dique, y por tanto su efectividad sobre estructuras ignotas, con posibles discontinuidades, oquedades o irregularidades.

Se optó por realizar una refundación parcial del dique, eliminando el relleno terrero y el inestable espaldón. No de una vez, sino poco a poco y por bataches, descubriendo en el proceso diversas estructuras murarías, canalizaciones en caja cerrada y canales (uno de madera), y otros elementos constructivos ocultos o enterrados, que posiblemente correspondan a anteriores fases de construcción o posiblemente a sistemas hidráulicos anteriores.

Las obras en el momento de terminar este documento se encuentran en fase de finalización.

Será con la redacción de la documentación de finalización de los trabajos, tras el estudio detallado y, análisis de todo el material recabado y documentación de obra, gráfica, anotaciones y fotográfica, y junto con el informe arqueológico, cuando se pueda determinar con precisión la naturaleza de los restos encontrados, su misión y su papel dentro del dique y en las posibles estructuras hidráulicas precedentes. Ha de quedar inevitablemente pospuesta su exposición.

Se están consultando diferentes fuentes y documentos con mención a conducciones de madera de los siglos XV y XVI, en concreto textos de María Isabel del Val Valdivieso, *"Musulmanes y cristianos frente al agua en las ciudades medievales"*

Inicialmente se realizo una cata en el tramo central del espaldón. No se localizó el muro de aguas abajo, análogo al del tramo norte que se esperaba encontrar, entendiendo que el espaldón lo estaba estabilizando, ya que se contaba con él en la idea de estábamos ante una presa a la holandesa. Ese muro no existía, y el pretil se apoyaba en el propio talud de tierra en casi todo su desarrollo, salvo en la zona contigua a la escalera, en que aparecieron unos muretes de mampostería, con taludes.

La obra se planteó por el extremo sur, desmontando el espaldón, siempre en taludes de poca pendiente. Así se fue procediendo, desplazando el tajo hacia el norte. A la vez se iba retirando la tierra de relleno hasta la hoja interior del muro doble de la pantalla.

Paulatinamente se excavaba hasta la roca y se alzaba un muro escalonado con contrafuertes o costillas transversales, ya que fue esta la solución adoptada conjuntamente.

Se desmontaría un tercio del dique, el de aguas abajo, constituido por el núcleo de tierras, y el espaldón o el muro de aguas abajo, según el extremo que corresponda a cada laso de la escalera central, que lo divide en dos partes. La obra se realiza pues por bataches, desde el extremo sur, eliminando poco a poco el espaldón y levantando un muro escalonado de refuerzo de aguas abajo, en toda la longitud del dique, unido a la escalera, que se desmontó y volvió a reconstruir de forma solidaria con él. Este muro presenta transversalmente tres escalones, dispuestos hacia el interior, de entre 2,2 y 2,5 m de altura, según tramos, con anchuras que van desde los 3,30, 2,20, 1,10 m progresivamente, con contrafuertes perpendiculares, de 2 m de anchura, separados 7,4 m a ejes, en el tramo sur, y más próximos en el norte, que entestan contra el muro interior de la pantalla de aguas arriba. Forman el muro escalonado y las costillas o contrafuertes, junto con la escalera, un conjunto o cuerpo único como muro de aguas abajo, manteniendo en cierta medida la idea original de doble muro, en este caso grueso, con relleno intermedio de tierras.

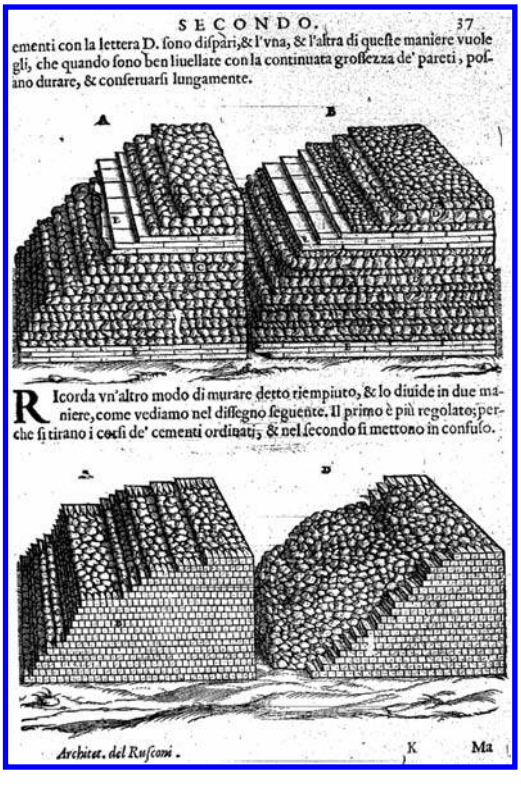
No se ha mencionado aquí que el lienzo de aguas arriba experimentó un refuerzo, con trasdosado de muros de mampostería de cal y canto y de bolos de cuarcita, de diferente grosor y altura, como recrecido de refuerzo en su engrosamiento, desde su base hasta dos tercios de su altura, aproximadamente, seguramente coincidiendo con las obras del s. XVII de terraplenado. Además en esa obras se condenaría el desagüe de fondo, situado en el cubo a unos 7 m de profundidad (siempre desde el nivel de la calle) y se abrirían unos desagües nuevos a media altura del Cubo, o se utilizaron los anteriores, uno o ambos, de medio fondo preexistentes,

que pasarían a ser los de vaciado de fondo, y que ahora se recuperan de forma efectiva.

Los paramentos vistos del nuevo muro o estructura muraría de aguas abajo y la escalera se ejecutan con hiladas de piedra recuperada, en ligero talud. Esta mampostería sirve de encofrado para ejecutar los escalones y costillas, que se van levantando por tongadas horizontales, evitando siempre las juntas verticales, dejando juntas de trabajo siempre oblicuas, y con tizones salientes. El material empleado es un hormigón ciclópeo con cemento puzolánico e hidróxido de cal, a la manera del *opus caementicium* romano, con áridos y rellenos (áridos gruesos), *caementa,* perfectamente caracterizados y determinados, con pruebas y ensayos en laboratorio en sus dosificaciones y resistencias.

Se consultaron con detenimiento diversos tratados y publicaciones específicas: *"La construcción romana. Técnicas y materiales",* de Jaen-Pierre Adam, *"L'art de bâtir chez les romains",* de Auguste Choisy y el *"Tratado de Arquitectura"*, sobre el Vitruvio, de 1590, de Giovanni Antonio *Rusconi.*

En los aspectos de ejecución ha sido fundamental la coordinación de la dirección facultativa con la empresa, y los diferentes y complejos trabajos de planificación de la obra de ésta, con ejecución


SECONDO. 37

ementi con la lettera D. ſono diſpari, & l'vna, & l'altra di queſte maniere vuole gli, che quando ſono ben liuellate con la continuata groſſezza de' pareti, poſſano durare, & conſeruarſi lungamente.

A B

Ricorda vn'altro modo di murare detto riempiuto, & lo diuide in due maniere, come vediamo nel diſſegno ſeguente. Il primo è più regolato; perche ſi tirano i coſti de' cementi ordinati; & nel ſecondo ſi mettono in confuſo.

C D

Architet. del Ruſconi. K Ma

Figura 20. Grabados del tratado de A. Rusconi. Muros de construcción romana.

in situ de los hormigones romanos, el acopio de los diferentes tipos de piedras, la cal, para las mamposterías y rellenos, el traslado de la tierra levantada o excavada, la ordenación de los tajos y tareas, especialmente de las cuadrillas de canteros, con sus maestros y oficiales y toda la concurrencia de diferente maquinaria, en ocasiones pesada.

Toda la fábrica levantada arranca sobre una cimentación, también en hormigón en masa ciclópeo, que asienta en todos los casos en la roca viva, con una cota de profundidad variable, hasta los 11 m, en un tramo.

Los cajones o senos huecos entre costillas y escalones se rellenan con terreno seleccionado extendido por tongadas y compactado. Estos cajones se encofran interiormente, junto con las caras interiores de las costillas con sistemas tradicionales de "peris". Las costillas se dejan 80 cm por debajo de la cota suelo acabado para trazar las conducciones enterradas correspondientes a las líneas de las infraestructuras, bajo la calle o "paseadero" del dique.

Se cuenta con un espacio vertical destinado a arqueta de llaves, maniobra y vaciado, entre dos costillas centrales, frente al cubo de desagües.

El conjunto está dotado con diversos drenes longitudinales y transversales, a diferentes alturas, con piezómetros de seguimiento y testigos visibles en la arqueta de llaves. Se mantiene el aliviadero del costado sur del vaso y se modifica, para ampliarlo, en un aliviadero de labio grueso el del extremo norte

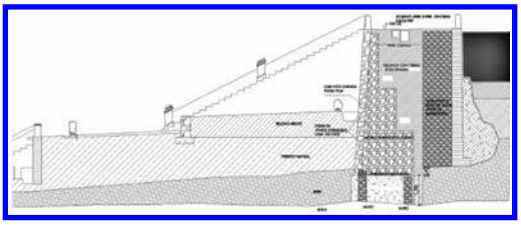


Figura 21. Esquema de sección transversal de un tramo tipo.

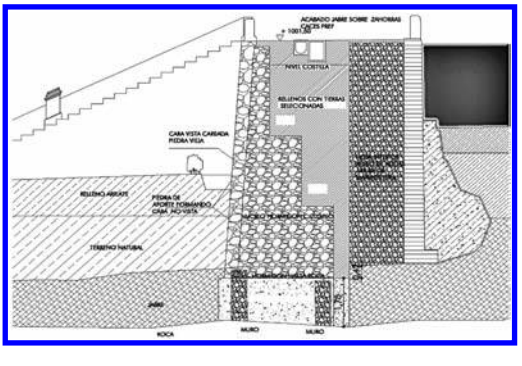


Figura 22. Detalle de esquema de sección transversal.

Figura 23. Vista general del estanque.

del dique, al comprobarse su insuficiencia. Se ha verificado numéricamente, bajo diferentes hipótesis de cálculo y comportamientos, que con los rellenos del núcleo central, las fábricas existentes del paramento aguas arriba y la nueva estructura muraria, se forma un conjunto, sólido, estable y coherente desde el punto de vista estructural y constructivo, y teóricamente desde el histórico.

Se pretende mantener por tanto el comportamiento hidráulico de la presa, sobre todo en relación al equilibrio de filtraciones y saturaciones respecto a las terrazas inferiores ajardinadas, con una construcción asimilable a los modelos holandeses de doble muro y relleno terrero, pero asegurando su estabilidad y funcionalidad. Se consigue, tras eliminar el espaldón, recuperar el sistema de terrazas renacentistas, con arriates ajardinados a pie de dique, y por tanto una composición espacial equilibrada, afín a la original, con la que se ha pretendido articular también un "proyecto de repristinación jardinera", remitiendo a un supuesto momento álgido en las grandes intervenciones en el Bosque, con el mayor rigor al que se podido llegar y en condiciones óptimas de seguridad.

En todo momento se ha seguido una metodología arqueológica exhaustiva, con seguimiento continuado, con desmontajes ordenados de piezas, con su numeración y siglado, para su recolocación en idénticas posiciones, tomas de datos y levantamientos gráficos de los restos y preexistencias localizadas, reportajes fotográficos e informe final, en fase de redacción.

Durante la ejecución se ha establecido un seguimiento de posibles movimientos del muro de aguas arriba, con precisión milimétrica, con dobles dianas superpuestas y otras en puntos fijos, en el lienzo de suso, con un sistema de estación total. Hasta el momento no se ha apreciado ninguno.

Con el desarrollo de las obras se requirió adaptar en parte las soluciones previstas a los condicionantes que se presentaron en el momento de ser ejecutadas, tanto en lo referente a las cotas del firme, a la aparición de de preexistencias, y a otras de carácter geométrico o constructivo, habituales en la práctica de la restauración.

Reiterar que se ha debido posponer, para una siguiente ocasión, la interpretación de los elementos, restos y fábricas encontrados, que podrán aportar quizá nuevos datos a la Historia de las Presas Históricas y a la práctica de su restauración.

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Dam Maintenance and Rehabilitation II – Romeo García et al. (eds)
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Presa de Ibaieder. Rehabilitación del drenaje

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ABSTRACT: Ibaieder is a rockfill dam 66 meters high with concrete screen; dedicated to water supply, was completed in 1991. The monitoring system allows tracking of water pressure at the foundation, seepage and strains. The deformational behavior has been always good, with small settlements and joints movements almost imperceptibles; leaks have been historically limited; uplift pressures, measured directly in the existing drains, have had high values. This water pressure and some loss of efficiency in drainage system recommended some works that would rehabilitate the drainage, detect the true state of the plinth's foundation and improve the pressure control system. The analysis, initiated during the execution of the work, leads to the conclusion that high pressures have fallen, waterproofing screen works properly and pressure control system has been improved.

1 INTRODUCCIÓN

Ibaieder es una presa de escollera con pantalla de hormigón, tiene 66 metros de altura sobre cimiento y 252 de longitud de coronación, está ubicada en el río Ibaieder y se dedica al abastecimiento. Se terminó en 1991 y retiene un embalse de 11,3 hm^3 (ver planta en Figura 9). El cimiento en la zona del plinto está formado por una sucesión de areniscas, calizas y argilitas meteorizadas en superficie y competentes en profundidad. Su sistema de auscultación permite el seguimiento de subpresiones, filtraciones y deformaciones, y sus datos han servido para la redacción de varios estudios de comportamiento.

El comportamiento deformacional ha resultado siempre bueno, como asientos pequeños de la escollera y movimientos en las juntas de la galería casi imperceptibles. Las filtraciones se han mantenido históricamente limitadas, sin dar motivo de preocupación al equipo de explotación. Las subpresiones medidas directamente en los drenes existentes han venido dando valores altos, muchos entre el 60% y el 80% de la carga de embalse, desde luego dependientes de la cota del embalse y bastante estables en el tiempo. La magnitud de la subpresión ha sido motivo de preocupación para el equipo de explotación, aunque limitada por la estabilidad en el tiempo de estos datos y por la falta de otras anomalías.

La toma de datos de subpresiones ha consistido siempre en obturar los drenes de una margen, esperar la estabilización y medir la presión de cada dren con un manómetro adecuado, dejando el dren ya abierto tras anotar la lectura. Después se procedía igual con la otra margen.

Este método de lectura, y sobre todo los valores de subpresión captados, introducían dudas sobre la situación hidráulica del plinto y un cierto temor a la existencia de fuertes gradientes de presión bajo su cimentación; nótese que en cada sección se disponía de un único piezómetro, momentáneo, por que enseguida volvía a ser dren, que no permitía captar cómo se disipaban las presiones altas medidas, antes de llegar a la escollera, totalmente permeable.

En el informe de comportamiento de 2007 se concluía, acerca del comportamiento hidráulico de la cimentación, lo siguiente:

- Las subpresiones medidas en los drenes de la galería son altas y con porcentajes, referidos a la cota del embalse, crecientes en los drenes más significativos; también se nota una clara dependencia entre las presiones medidas y la carga de embalse, lo que significa que hay una conexión buena entre el embalse y la zona de la cimentación situada inmediatamente debajo de la galería, y ello a pesar del teórico largo camino de las líneas de filtración, que tienen que recorrer el tapiz de arcilla que recubre la zona de aguas arriba, delante del plinto y la propia cimentación de la galería, antes de llegar al dren-piezómetro (ver Figura 1).
- Hay que tener en cuenta que el análisis que se ha hecho de las subpresiones, considerándolas

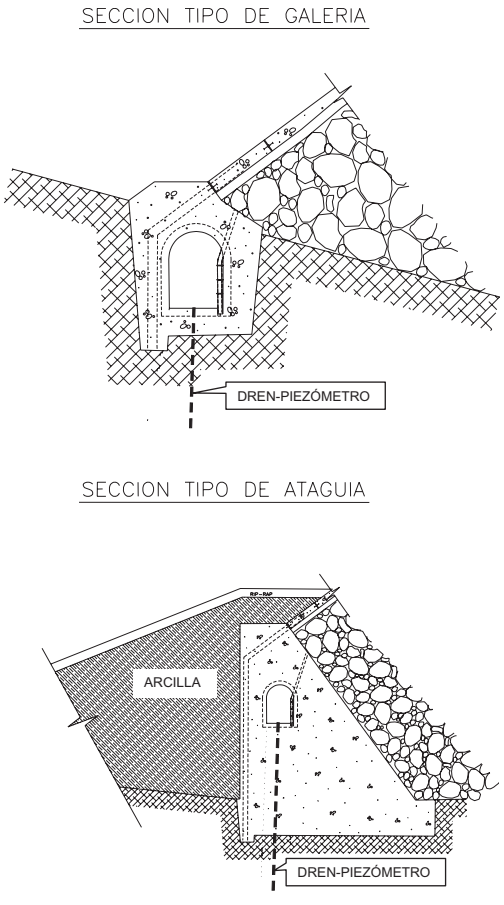

Figura 1. Secciones tipo por galería y plinto.

en porcentaje para independizarlas en lo posible de la cota de embalse y refiriéndolas al fondo del taladro, proporciona una visión muy realista de su verdadera magnitud.

– Por otra parte, el corto camino existente entre la zona de los drenes-piezómetros, inmediatamente debajo de la galería, y la escollera confiere a ésta zona de cimentación unos gradientes hidráulicos potencialmente muy fuertes y nada convenientes (puede contemplarse este aspecto en la Figura 1, sacada del informe de comportamiento de 1997).
– De ello se deduce una necesidad de mejorar el drenaje con el objetivo de bajar las presiones y mejorar el gradiente.
– Los caudales en los drenes no son grandes y además están en un proceso decreciente con el tiempo, como puede observarse en la Figura 2, sacada del informe de comportamiento de 2007, que muestra las filtraciones medidas frente a la respectiva cota de embalse, superponiendo la

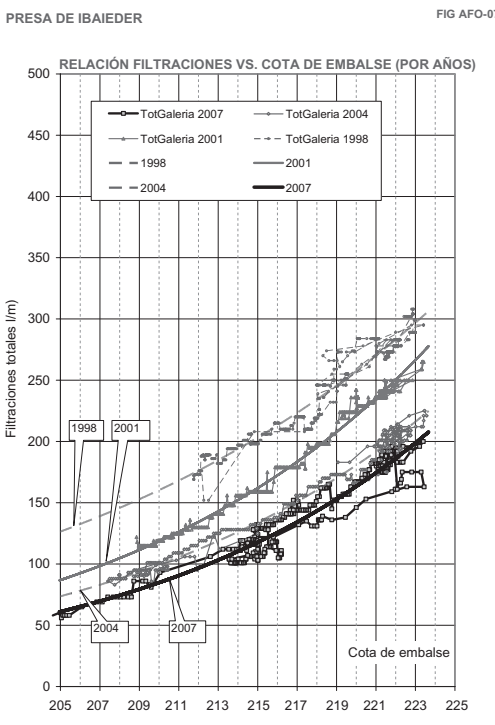

Figura 2. Disminución de las filtraciones totales con el tiempo, más notables en las curvas de tendencia superpuestas.

tendencia de cada año. Resulta muy recomendable el acometer una actuación de mejora del drenaje, probablemente aprovechando los actuales drenes para impermeabilizar y realizando después otros nuevos.

Con este panorama de cierta pérdida de eficacia en el sistema de drenaje, y de confirmación de altas subpresiones, más preocupantes por no conocerse su distribución en dirección de aguas arriba hacia aguas abajo, se acometió la ejecución de un proyecto que debía rehabilitar el drenaje, detectar la verdadera situación hidráulica de la cimentación del plinto y establecer un sistema de control de subpresiones que no interactuase con el sistema de drenaje.

2 PROYECTO DE REHABILITACIÓN

2.1 *Consideraciones previas*

En el proyecto de rehabilitación se hacen consideraciones sobre lo recomendado en el informe de 2007 (el inmediatamente anterior en el tiempo), siendo de particular interés la que se refiere al drenaje: *"Estudiar una actuación que lleve a mejorar el drenaje en la*

zona de cimentación del plinto. Esta actuación podría consistir en una limpieza, o reperforado, de drenes que no parece pueda ser suficiente si se tiene en cuenta que hace años que se tienen presiones altas, es decir, no todo es pérdida de eficacia del sistema de drenaje. Parece necesario practicar nuevos drenes, al menos en las zonas históricamente con mayores presiones. Utilizar los drenes actuales para una labor de impermeabilización, que deberá ser muy controlada, y perforar nuevos drenes, probablemente más y dispuestos para poder captar el gradiente hidráulico en algún sitio, puede ser la mejor alternativa."

También se llama la atención sobre las particularidades de la cimentación de la presa. En los párrafos siguientes se recogen los aspectos de mayor interés:

- Las litologías sobre las que se apoya la presa parecen, en principio, bastante impermeables; son margas, areniscas y argilitas secundarias. A su vez, la estratificación, paralela al cauce y según planos normales a la ladera izquierda, debe considerarse una suerte…
- Ha de observarse que prácticamente la totalidad de la presa apoya sobre margas y areniscas y es muy probable que sea en los planos de sedimentación donde tenga lugar el desarrollo de presiones intersticiales importantes: las areniscas, a través de su red de fisuración, aportarán algo de agua, no tiene porque ser mucha, las margas actuarán de barrera impermeable y el incremento de la carga, al ascender el nivel hidrostático, hace el resto.
- El incremento de presiones de agua, observado a través de los drenes convertidos en piezómetros, se halla ligado al ascenso del nivel del embalse, con entrada principal a través de las areniscas, confinamiento aportado por las margas y manifestación actuando contra la cimentación de la galería y plinto. El alargamiento de las líneas de filtración, que debiera haber aportado el manto de arcilla situado aguas arriba, contra el plinto, no está teniendo la efectividad deseada, al menos eso se deduce de las altas presiones encontradas.
- Por otra parte, la tipología adoptada para el plinto (perfil de gravedad) tampoco ayuda mucho, ya que, a embalse casi lleno, con poca carga trasmitida por la pantalla y escollera, se estarán produciendo extensiones en la zona de cimentación, aguas arriba (con aperturas para el agua), y compresiones en la misma zona, aguas abajo (ver Figura 1).
- La limpieza de los drenes, que podrán estar bastante carbonatados por aportaciones procedentes de las margas, es una mejora indudable, aunque el incremento de su efectividad, con nuevos drenes, resulta también necesario.
- Cabe dudar algo de la efectividad de unas inyecciones realizadas sobre el contacto hormigón-cimentación, por ello convendrá aplicarlas sólo allí donde hay caudales notables.

2.2 *Solución proyectada*

La solución proyectada se extendía por cuatro apartados.

- Mejora del drenaje actual consistente en realizar una profundización de drenes, particularizada para cada zona de la galería, dentro de la cimentación, a la vez que una limpieza de las obturaciones que se encuentren.
- Inyectar determinados drenes (19 de ellos previstos en proyecto), que se especificarían definitivamente en la fase de obra, para frenar lo que se considere un caudal de filtración alto. Estos drenes inyectados debían ser compensados por otros nuevos, situados en sus inmediaciones.
- Aumentar el drenaje existente, teniendo en cuenta las particularidades del terreno de cimentación, las conocidas y las que iban a deducirse durante el desarrollo del trabajo. Esta mejora del drenaje iba a estar supeditada a los resultados obtenidos en la limpieza, aunque en el proyecto se proponían ya un total de once.
- Instalar piezómetros eléctricos, de cuerda vibrante, que debían permitir un mejor conocimiento de la situación de subpresiones en cimentación, incluyendo en ese conocimiento, la manifestación de gradientes de subpresión, es decir, para saber cómo se disipa la subpresión a medida que se avanza por las líneas de filtración. Para ello se proponían ocho secciones instrumentadas con piezómetros de cuerda, instalando un mínimo de dos piezómetros por sección y un máximo de cuatro (ver Figura 7). Los nuevos piezómetros se recogerían en una unidad de adquisición de datos automatizada y el sistema debía permitir un control de resultados durante la ejecución del drenaje.

Como es lógico, los trabajos de inyección debían preceder a los de limpieza de drenes o ejecución de unos nuevos y la instalación de piezómetros debía estar entre la inyección y el drenaje, para sacar la mayor información posible del avance de este último.

3 TRABAJOS DE REHABILITACIÓN

Los trabajos se iniciaron a principios de febrero de 2009, con la impermeabilización para evitar que trabajos de inyección pudieran afectar a los drenes previstos, obturándolos, o a los piezómetros a instalar que podían resultar averiados. La actuación empezó con dos drenes de la margen derecha, en

la parte alta de la galería, que se consideraban fáciles, por su mediano caudal aportado y por su accesibilidad. Los resultados se aprovecharían luego para decidir sobre la continuación de los trabajos.

3.1 *Reconocimiento y primeras inyecciones*

Como alternativa a lo propuesto en el proyecto (inyección directa del dren con lechada de cemento), se adoptó la solución, propuesta por la contrata (HCC), de realizar cuatro taladros delante del dren, separados del orden de un metro entre sí, de la misma profundidad del dren, e inyectarlos con resina epoxi, de fácil control en cuanto a limitar el alcance que podría tener la zona inyectada, también por la seguridad que proporcionaba el método frente a agresiones al medio ambiente, fundamentalmente salidas de cemento hacia el embalse.

Teóricamente, podía esperarse que los taladros para impermeabilizar cada dren tuvieran una mayor aportación de caudal que el propio dren a impermeabilizar—aportando preferentemente por la zona de contacto galería-roca—y que la impermeabilización dejase el dren útil, con un caudal claramente menor, incluso sin que la resina llegara a afectarlo. Con estos resultados esperados, se podría aumentar después el drenaje para aliviar subpresiones.

En la realidad, con la apertura de los taladros aguas arriba del dren 30D (ver Figura 7) se rebajó su caudal aunque las aportaciones de los nuevos taladros eran superiores a las del dren. La aportación de caudales se ubicaba claramente dentro de la cimentación estando el contacto hormigón-roca bastante impermeable.

En el dren 24D, se repitió la situación del anterior, pero con la variante de que los nuevos taladros aportaban caudales grandes, de varias decenas de litros por minuto. Las aportaciones venían también por la roca, captadas en varias franjas más permeables. En un intento de localizar y delimitar una vía preferente de filtraciones se aumentaron los cuatro taladros inicialmente previstos, a uno y otro lado del dren, siempre por aguas arriba del mismo, hasta completar un total de 11 taladros, resultando todos de similares características de permeabilidad: entrada de agua preferentemente por franjas más permeables de la cimentación.

Los testigos obtenidos presentaban un hormigón de características mecánicas adecuadas, con presencia escasa de macroporos o pequeñas coqueras. La unión hormigón roca estaba aparentemente sana y la roca presentaba bastante hetero geneidad o, dicho de otra manera, estratificación. Se identificaron areniscas gris oscuro, relativamente fracturadas y argilitas entre fracturadas y muy fracturadas.

Se introdujo una cámara de vídeo en varios de los taladros realizados. El análisis de los videos mostró zonas de la roca donde la superficie cilíndrica de perforación resultaba perfecta, como corresponde a una roca compacta y resistente y otras, intercaladas con las anteriores, frágiles y rotas; por estas últimas se producía la mayor parte de la aportación de agua. En ocasiones, el agua arrastraba pequeños fragmentos de roca.

En la situación anteriormente presentada, se acometió la inyección de los cuatro taladros del dren 30D. El proceso se realizó de forma sucesiva, empezando por el taladro más bajo y próximo al centro de la presa. Siempre se trabajó con los taladros que todavía no estaban inyectados abiertos, con excepción del cuarto y más alejado del centro de la presa, que tuvo conectado un manómetro; el dren 30D estuvo también siempre abierto.

Las presiones ejercidas para introducir la resina en los taladros se mantuvieron casi constantes en todo el proceso, desde el inicio, cuando sólo se rellenaba al taladro, hasta el final, cuando se decidió no inyectar más material. Se notó un cambio leve al aumentar la viscosidad de la resina, este cambio se puede estimar entre un 10 y un 20% de sobrepresión. Se emplearon resinas de baja viscosidad, de media y alta, terminando siempre con alta viscosidad, con la excepción del cuarto taladro. Las presiones finales en la zona inyectada no se pudieron conocer, sí se comprobó que en el manómetro colocado en la boca de uno de los taladros no apreció presión. Se puede resumir por tanto, que en el proceso no hubo que forzar la presión en modo alguno para inyectar el volumen de resina utilizado.

Como resultado final, un caudal inicial del dren 30D de unos 8 l/m quedó en unos 9 l/m, es decir, levemente aumentado.

Se notó también que el color del agua drenada experimentaba un enturbiamiento hasta llegar a un color gris oscuro cada vez que se acometía la inyección de un nuevo taladro; ello se notaba en el taladro siguiente, todavía sin inyectar. El dren dio siempre agua limpia.

Con ello se concluyó que el efecto de la resina sobre las franjas inyectadas era de abrir levemente los diaclasados existentes favoreciendo la entrada de agua a su través y provocando también leves arrastres. Se esperaba una recuperación rápida de estas aperturas, volviendo el terreno a cerrarlas comprimiendo la resina todavía muy flexible, y, en efecto sucedió, pero de forma limitada, ya que 48 horas después de la inyección el caudal sólo bajó a 5 l/m.

En cuanto a la viabilidad de emplear resinas para mejorar la impermeabilidad, se concluyó que las vías de agua resultaban abiertas por las presiones puntuales, pero altas comparativamente, de las

resinas y que ésta no iba a resultar adecuada, salvo si se empleaban volúmenes altos con viscosidad muy baja, posiblemente con presiones mayores, que exigirían grandes cuidados—peligrosas por tanto—, siempre con pruebas previas de la efectividad del proceso. En nuestro caso, esta alternativa resultaba descartable ya que no se pretendía una impermeabilización notable y no se tenía prevista una inversión acorde, considerada, por otra parte, de interés secundario.

También como resultado de estos primeros trabajos de impermeabilización se concluyó que la pantalla de impermeabilización de la presa tiene un alcance más limitado que el supuesto, en cuanto a su extensión hacia agua arriba. Para comprobar este punto, se ejecutaron dos taladros de reconocimiento en coincidencia con el dren D30, una de ellos vertical y otro inclinado unos 15 grados hacia aguas abajo. El taladro vertical mostró signos de lechada de inyección metida en la época de construcción, pero en menor cantidad que la que cabía esperar. El taladro de aguas abajo no presentó signos de lechada y tuvo el efecto de quitar caudal al nuevo taladro vertical, precisamente en el nivel del contacto hormigón-roca, donde ambos están muy cercanos.

Como resultado de las inyecciones realizadas y del reconocimiento de la cimentación obtenido con los correspondientes taladros, sus testigos y la propia inyección, se abandonó la alternativa de realizar inyecciones con resina, volviendo al empleo de lechada de cemento y a unas prescripciones que limitaban la presión de inyección a 1,3 veces la carga del embalse sobre la boca del taladro, en el momento de inyectar.

3.2 *Inyecciones, drenaje y piezometría*

Se inyectaron los drenes originales que proporcionaban más caudal y fueron sustituidos por otros nuevos, todo ello según estaba previsto en el proyecto.

En cuanto a la ejecución de nuevos drenes, se confirmaron los propuestos en el proyecto mediante inspecciones en la galería para anotar los drenes inefectivos (aquéllos que prácticamente no goteaban). Estos coincidían con puntos de presión baja, medida a través de los drenes. Es en estas zonas donde se incidió con nuevos drenes. Esta alternativa se estimó más efectiva que la de limpiar los drenes no efectivos.

Se perforaron los taladros para alojar los piezómetros. En la Figura 3 siguiente se muestra la ubicación esquemática definitiva de los taladros hechos para ubicar los piezómetros de cuerda vibrante instalados en la galería. Cada piezómetro va situado inmediatamente debajo del contacto entre el hormigón y la cimentación.

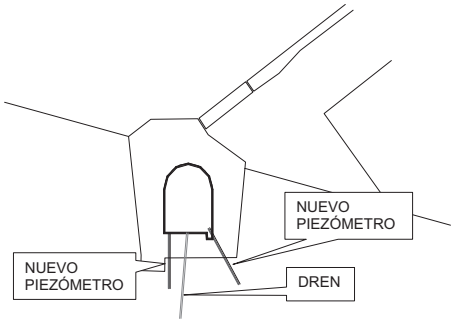

Figura 3. Situación de los piezómetros de cuerda vibrante en la galería.

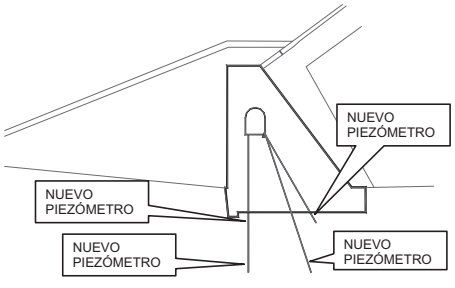

Figura 4. Situación de dos niveles de piezómetros de cuerda vibrante en el plinto.

En la Figura 4 siguiente se puede ver el mismo esquema, para los piezómetros ubicados en el plinto. Sólo hay dos secciones, las correspondientes a los drenes-piezómetros originales 6D1D y 13D2I, con piezómetros en dos niveles. Los piezómetros van alojados en los taladros en una profundidad aproximada indicada por la flecha en la figura. Nótese que el dren indicado en la Figura 3 está en distinta sección que los piezómetros.

3.3 *Perforación de nuevos drenes*

Se perforaron los nuevos drenes en paralelo con la instalación de los piezómetros cuando este trabajo no afectaba a los nuevos taladros o después de inyectar aquéllos, cuando los nuevos drenes estaban muy próximos a los piezómetros de cuerda.

3.4 *Subpresiones en los piezómetros nuevos*

Con la instalación de cada piezómetro se empezaron las campañas de lectura que se regularizaron cuando se contó con el nuevo sistema de adquisición de datos automatizado, a mediados de abril de 2009.

En la Figura 5 siguiente se muestra el estado de subpresiones medidas en la presa el día 31 de

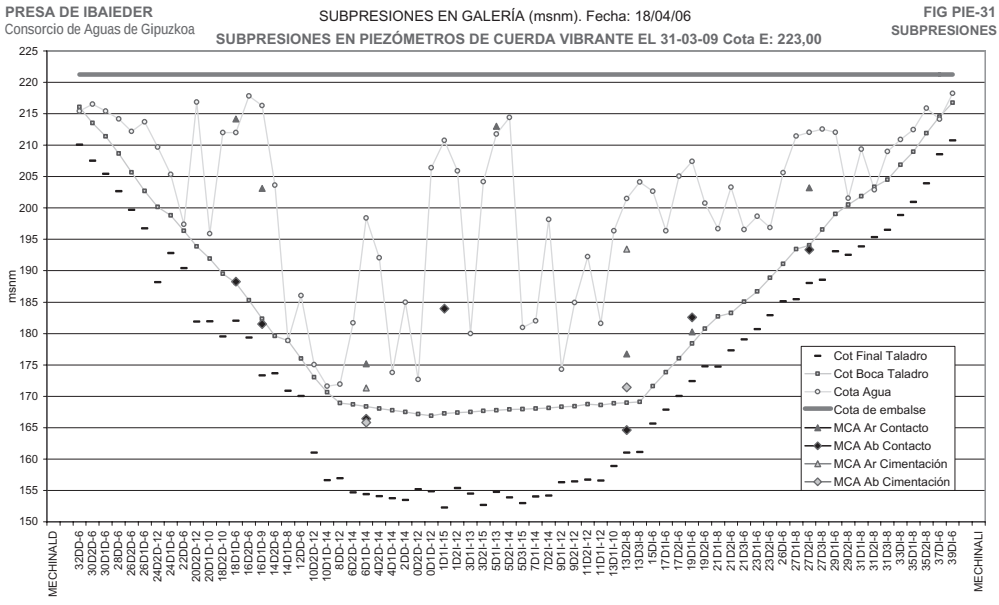

Figura 5. Subpresiones en drenes y piezómetros de cuerda vibrante comparadas.

marzo de 2009, a primera hora de la tarde. La línea clara con pequeños círculos representa las subpresiones medidas el 18/04/06, con la presa a una cota de embalse similar a la del 31 de marzo. Los triángulos oscuros representan la cota alcanzada por el agua en cada uno de los piezómetros situados más aguas arriba y en el contacto hormigón-terreno; los rombos representan la misma cota de agua, pero para los piezómetros de más aguas abajo (ver situación en Figura 4). Puede notarse que las secciones con piezómetros de cuerda están distribuidas en zonas que han venido presentando presiones altas medidas en los drenes.

Algunos de los nuevos piezómetros de aguas arriba presentan presiones altas muy similares a las captadas en los drenes, concretamente en 18D1D y 5D1I. Además, aunque la fecha de medida está muy cercana a la de instalación de piezómetros e incremento de drenaje, las presiones medidas entonces en 16D1D, 13D2I y 27D2I, altas pero menores que las antiguas, estaban ya rebajadas por la perforación de los nuevos drenes.

Había otras zonas: 6D1D, 1D1I y 19D1I que presentaban presiones bastante menores que las obtenidas midiendo en los drenes.

Se concluía en aquellas fechas que las presiones altas medidas a través de los drenes eran reales y representativas de la situación normal de la presa, pero ello sólo en algunas zonas, mientras que, en otras, era el propio sistema de medida el que producía presiones altas que bajaban en cuanto volvía a funcionar el drenaje.

Gracias a la instalación de dos piezómetros en cada sección, uno más hacia aguas arriba que el otro, podían constatarse caídas de presión en cada sección, entre cada piezómetro de aguas arriba y el correspondiente de aguas abajo. Estas caídas eran muy notables cuando el piezómetro de arriba marcaba presión alta. Este comportamiento hidráulico de la cimentación, sólo constatado a través de los nuevos piezómetros, proporciona mucha tranquilidad respecto al funcionamiento seguro de esta zona de la presa, tan crítica.

Esta mayor tranquilidad se explica en la Figura 6 siguiente, donde se representa, con flechas, una

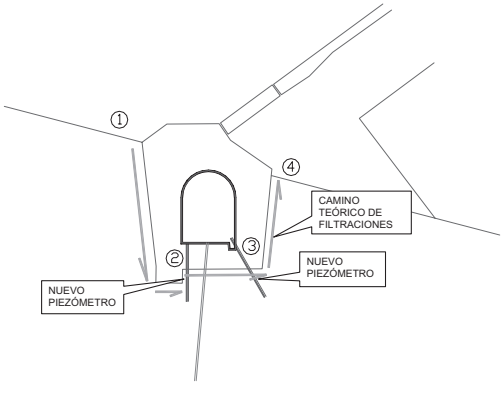

Figura 6. Camino teórico de filtraciones alrededor de la galería.

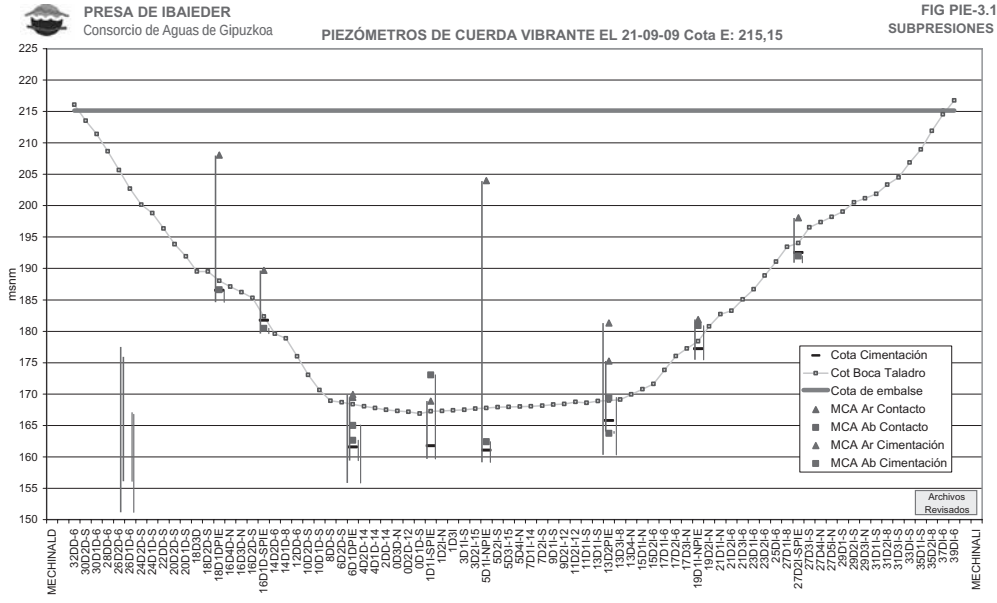

Figura 7. Subpresiones en piezómetros de cuerda vibrante en septiembre de 2009.

trayectoria teórica del agua de filtraciones caminando desde el embalse (1) y circulando a lo largo del contacto hormigón-terreno para pasar junto al primer piezómetro (2), junto al segundo (3) y saliendo hacia el cuerpo de presa que lo drenará hacia el cauce aguas abajo.

En un planteamiento muy teórico, la presión de agua pasaría de ser la del nivel del embalse en (1) hasta cero, o casi, en (4). La pantalla de impermeabilización y el drenaje pretenden que la presión baje mucho en el paso entre (2) y (3). Está claro que, si la presión que ejerce el embalse entrase hasta (3) o más allá, acabaría rompiendo hidráulicamente el terreno y creando una vía de agua peligrosa para la seguridad de la presa.

Por lo dicho, que exista una gran diferencia entre las presiones de (2) y de (3) es muy bueno; ello llevará, además, a que haya una baja presión en (3). Los nuevos piezómetros están estratégicamente situados para comprobar este buen funcionamiento de la cimentación o detectar lo contrario.

4 RESULTADO FINAL

En la Figura 8 siguiente se muestra la situación de presiones para la última fecha analizada, el 21 de septiembre de 2009.

Como se ha dicho, los triángulos representan la subpresión en los piezómetros de aguas arriba próximos al contacto, expresada en metros de carga de agua y dibujada en la misma escala vertical que tiene la presa y el nivel de embalse (horizontal aproximadamente en la cota 215). Ahora son los rectángulos quines representan la subpresión en los piezómetros de aguas abajo próximos al contacto. Triángulos y rectángulos claros corresponden, respectivamente, a piezómetros de arriba y de abajo situados más profundos dentro de la cimentación. Las líneas verticales van desde la posición de ubicación de cada piezómetro hasta la cota que alcanza su presión de agua.

Puede notarse que la presión mayor corresponde al piezómetro de aguas arriba de la sección 05 que, en contraste, tiene su piezómetro de aguas abajo casi sin subpresión. En el piezómetro de aguas arriba el porcentaje es del 80%, es decir, muy directamente conectado con el embalse.

La siguiente máxima presión alcanzada por el agua en un piezómetro corresponde al de aguas arriba de la sección 18. Aquí también sucede que el piezómetro de aguas abajo apenas tiene subpresión. Algo parecido puede decirse de las secciones 16, 13 y 27, todas con presiones pequeñas aguas arriba y casi nulas aguas abajo.

Dejando aparte las secciones 01 y 27, puede notarse que, en los piezómetros de aguas abajo, la subpresión ni siquiera da para llegar a la boca del taladro, si este estuviese abierto, como los están los drenes. La sección 19 es una excepción poco notable; aquí ambos piezómetros dan presión baja, pero suficiente para superar la cota del suelo de la

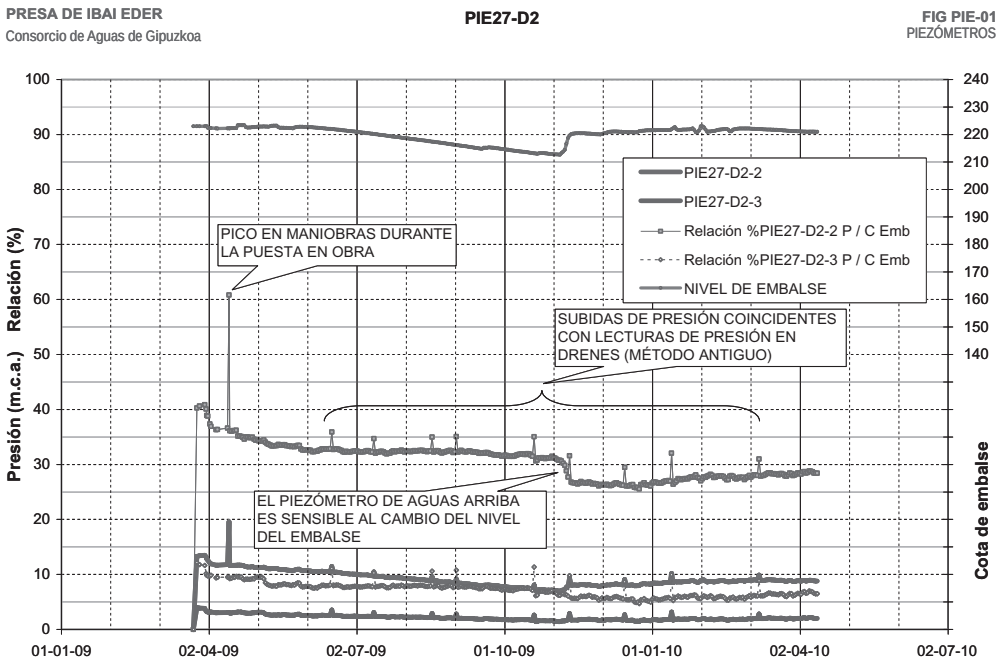

Figura 8. Evolución de subpresiones en una sección.

galería. Cabe notar que, en porcentajes, apenas se llega al 17%, cifra esta bastante baja.

La sección 01 presenta la curiosidad de que la subpresión es mayor en el piezómetro de aguas abajo que en el de aguas arriba. En cualquier caso, en ella las presiones aguas arriba llegan al 16%, siendo aguas abajo sólo del 24%.

Con datos de septiembre de 2009 se establecieron las siguientes conclusiones sobre la mejora del drenaje realizada:

- Las mejoras del estado de subpresiones introducidas por el drenaje han sido significativas. Ello se notó ya inmediatamente después de la perforación de los drenes gracias a la pronta instalación de los piezómetros.
- Sólo quedaron dos valores de porcentaje de subpresión—referido a la cota del embalse sobre los piezómetros—que superaban el 50% y correspondían a piezómetros de aguas arriba; sus respectivos de aguas abajo no superaban el 10%. El siguiente mayor es también de aguas arriba, pero profundo y no llegaba al 40%. Todos los demás estaban por debajo del 30%.
- La situación de subpresiones resultante se consideraba muy satisfactoria y adecuada para el funcionamiento de la presa.

También se concluía que la impermeabilización realizada—considerada tarea secundaria en el proyecto, dado que el caudal de filtraciones existente en la presa era moderado y viniéndose a menos—limitada en sus objetivos, fue conveniente ya que, con sólo un 16% de incremento de caudales filtrados en la galería, se habían bajado notablemente las subpresiones.

Además, el nuevo control de filtraciones tiene una doble ventaja sobre el anterior: por una parte da información de la situación normal de funcionamiento de la presa, el anterior mostraba subpresiones mayoradas, al cerrarse el drenaje para medir; por otra, da una nueva información, tanto o más importante que la de la subpresión, que es la que corresponde a la caída de presión entre el piezómetro de aguas arriba y el de aguas abajo. Cuando la caída de presión existe, aunque haya presiones altas, la resistencia hidráulica del terreno también existe y, si el segundo piezómetro tiene presiones bajas, como pasa en Ibaieder, la situación resulta muy tranquilizadora.

Los trabajos fueron realizados por HCC (Hidráulica, Construcción y Conservación), sus técnicos aportaron alternativas de interés y una buena labor de control.

5 RESULTADOS VISTOS UN AÑO DESPUÉS

La imagen que daba la presa, en cuanto a subpresiones, en septiembre de 2009, se mantiene desde

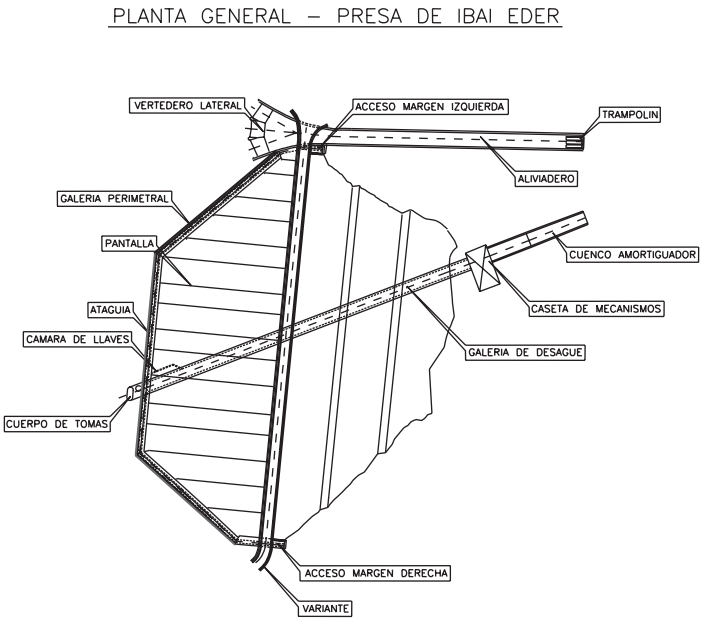

Figura 9. Planta general de la presa.

entonces. En un análisis realizado sobre datos recientes, puede notarse lo siguiente de mayor interés:

- Algunos de los piezómetros situados más aguas arriba, los denominados 2, se muestran sensibles a la variación del nivel del embalse (en la Figura 8 puede verse un ejemplo), manteniendo dos de ellos las presiones altas después de mejorar el drenaje.
- Por el contrario, los piezómetros de aguas abajo, los denominados 3, no son afectados por el nivel del embalse, o lo son en muy leve magnitud.
- Con la excepción de la sección de los piezómetros 19, donde la subpresión es muy pequeña, no se alcanzan los 10 metros de carga de agua, la bajada de presión que hay entre el piezómetro de arriba (2) y el de aguas abajo (3) es notable. Esto se nota más cuando se mira a los porcentajes (ver Figura 8).
- En todos los piezómetros se nota, en mayor o menor grado, cuando se cierra el drenaje para medir las subpresiones en los drenes por el método antiguo. Los valores de picos mostrados en la Figura 6 están minorados por razones debidas al método de presentación de datos (sólo se presenta una lectura promedio diaria).

6 CONCLUSIONES

Las mejoras del estado de subpresiones introducidas por el drenaje han sido significativas. Ello se notó ya inmediatamente después de la perforación de los drenes gracias a la pronta instalación de los piezómetros; en ocasiones pudo notarse como bajaba la subpresión a medida que se profundizaba en la creación de nuevos drenes; también el efecto contrario si se obturaba uno de estos drenes en tareas de comprobación.

Sólo hay dos piezómetros cuya subpresión supera el 40% de la carga de embalse que hay sobre la cota del piezómetro (piezómetros 2 de las secciones 18 y 05), corresponden a piezómetros de aguas arriba; sus respectivos de aguas abajo siguen sin superar el 10%. Por ello, debe considerarse la situación actual de muy satisfactoria y adecuada para el funcionamiento de la presa.

Comparando esta situación con la que había en la presa antes de la rehabilitación, hay que destacar que antes la mayor parte de los piezómetros superaban el 40%, con un promedio del 58%. De todas formas, esta comparación está muy distorsionada por el hecho de que las medidas antiguas aumentaban las subpresiones cuando se obturaba para medir.

Los trabajos de rehabilitación realizados se han apoyado mucho en el control de presiones, tanto las de inyección como las de presión de agua en cimentación. Gracias a ello y a las inspecciones realizadas durante los trabajos, se han podido afinar mucho los trabajos y obtener de ellos un buen grado de conocimiento sobre su desarrollo y sobre la manera de comportarse de la presa.

En cuanto al método antiguo de medida de subpresiones, todavía relativamente extendido en

muchas presas, tanto de gravedad como de materiales sueltos, hay que deducir que no es conveniente. Por una parte distorsiona las presiones obtenidas, en general mayorándolas, salvo que con caudales pequeños no se dé tiempo al manómetro a alcanzar su máximo, por otra somete a la presa, durante el tiempo de adquisición de datos, a sobrepresiones no deseables. Como paliativo de estas desventajas, hay que contar con una organización de las lecturas donde siempre se repite idéntico proceso, en caso contrario sería mucho mayor la distorsión, y con la habilidad del técnico que las interpreta.

Las pruebas de inyecciones con resinas epoxi, que tan buenos resultados han dado en otras ocasiones, se mostraron aquí ineficaces y muy probablemente contraproducentes.

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Monitoring seepage anomalies in the Jaburu I Dam

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ABSTRACT: The Jaburu I Dam can be pointed out as a classical case of safety dam granted by continuous monitoring and remedial process intervention. Since its first fulfillment, five years after its construction, a significant amount of anomalies associated to seepages through the foundation and the embankment has been recorded. The main event occurred in 1983 when the discharge on the left abutment reached 47 *l*/s and was followed by others, secondary in magnitude of discharge but, as dangerous as the first one concerning to the dam safety. Along the period of operation of the dam, seepage appeared also in the foundation and through the dam, events were monitored by flow gauges distributed along downstream toe of the dam. This paper presents the interventions adopted to release the seepage, comprehended grouting with cement and asphalt on the left abutment, grouting with bentonite on foundation, evaluation of the dam safety considering the collapse of the internal drainage system, reinforcement of drainage system and replacement of parts of the embankment due to the over settlement.

1 INTRODUCTION

The Jaburu I dam is located at Ceará State, Northeast of Brazil, and has shown anomalies in its drainage system. These anomalies caused seepage through the embankment and foundation. The evolution of the problem has been monitored by the Water Resources Management Company of State of Ceará (COGERH) according to the procedures recommended by Record's Inspections of Dams and Reservoirs of the Ministry of Integration, Brazil (Brasil, 2005).

The dam was constructed upon complex sediment corresponding to cuestiforme plateaus dominated by sandstone rock, intercalated with conglomerate of shale, claystone and siltstone.

The reservoir was completely filled up in 1983 when it reached the maximum regular water level. After five years of operation an increased amount of seepage has been recorded, initially in the left abutment, later in the downstream slope of the dam, at various locations. Its is known (Ceará, 2010) that when the reservoir reached its higher levels, seepages usually appeared somewhere, being the more significant ones next to the outlet works, at the maximum section (next to the central valley) and on left abutment. Discharges were observed in some points of the final part of the downstream slope.

Planned actions were proposed to reduce discharges after their establishments. From October 1989 to February 1990 one was able to observe the first remedial campaign focused in the seepage detected in 1983.

Nowadays the monitoring process is more efficient than 20 years ago, once the water management in Ceará state was formally structured since 1994, after the legal support State Plan of Water Resources coordinate by Water Resources Secretary-CE (Ceará, 1996).

The regulatory low for dam safety in Brazil is getting a new tool to due the process on monitoring dams. It would consist of a continuous process of inspections, diagnoses and remedial anomalies. Therefore a normal technical proceeding will be soon a duty for all owners and manager of dams. (Brasil, 2009).

2 JABURU I DAM

2.1 *Location of the dam*

The Jaburu I dam is located on the Jaburu River, with geographical coordinates 3°49′59″S, 41° 6′15″W, in the far Northwest of Ceará state (Fig. 1), in the region of Ibiapaba Plateau, comprehended by the districts of Tianguá and Ubajara, and affect over 160,000 inhabitants. The main water demand is agricultural irrigation, followed by human consumption of the urban and rural communities.

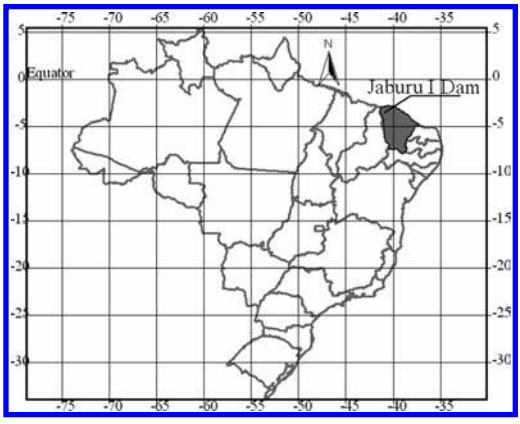

Figure 1. Location of the dam in Brazil.

Figure 2. Aerial view of the dam with the spillway.

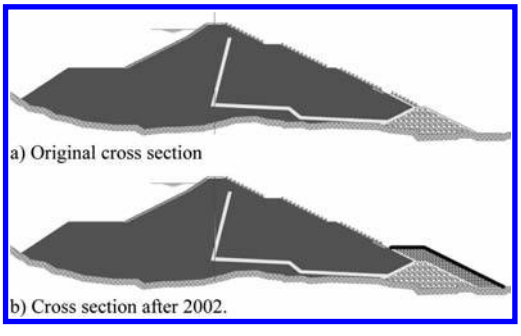

Figure 3. Typical cross sections of the Jaburu I Dam.

2.2 *Characteristics of the dam*

The dam was constructed within the period of 1981–1983, with 48 m of maximum height (726.10 m) and 770 m long, draining an area of circa 316 km². It has a reservoir with a storage capacity of 210 hm³. The designed drainage system was constituted by conventional vertical and horizontal filters, discharging through a toe rock-fill.

The main structure related in this paper is the earth embankment constructed with argillaceous soil material described as laterite (Fig. 3a). The embankment was constructed directly upon rocky stratum of sandstone presented strongly diaclased, clear coloration, intercalated for shale, siltstones and caulinítica stratum, sub-horizontal plans distributed in innumerous layers.

The spillway is located on the right abutment and is constituted of a canal excavated in sandstone rock with 29 meters of width and control structure in low ogive, designed for a discharge of 155 m³/s (Fig. 2). The outlet works is a gallery type, with pipe with 600 mm of diameter and extension of about 70 meters.

2.3 *Historical of anomalies*

The Jaburu I dam was constructed during the period of 1981–1983, and the first fulfillment occurred in 1983. Since then an amount of incidents occurred in the dam structure, according to the records described (Ceará, 2010):

– At about 5 years after the first fulfilling, in 1988, the seepage was identified in the left abutment, about 30 meters distant of the rock toe offset In the occasion, it was registered a discharge of 47 *l*/s, enough to form a cave at level of 700.00 m, 26.10 m inferior to the crest level. Beyond the main discharge recorded next to the left abutment, a secondary event of seepage was observed aside the outlet work. The crest of the dam presented also anomalies such as cracks along and in its transversal axis, and an over settlement in the upper layer of the embankment reaching up to 0.50 m.;
– In 1992 signals of seepage were detected downstream the dam, at the region between the outlet works and the center of the valley. The discharge all together registered 26 *l*/s;
– In 1997 various signals of seepage appeared on downstream slope, characterized by humid zones, mainly in the central valley, suggesting low performance of internal drainage system;
– In the rainy season of 2009 a significant increase of the seepage through the foundation was observed, mainly, in the area next to the left abutment, within a range between 3.33 *l*/s and 6.14 *l*/s. The critical event occurred at 08/20/09, when it was recorded the presence of residues of asphalt proceeding from the left abutment, which could be associated to the processes of internal erosion in the treatment of left abutment

occurred in 1989. After the humid period it was verified a reduction of the seepage in the same proportion of the reduction of the water level in the reservoir, and the flow steadied itself at circa 3.42 *l*/s;
– Recently, in December 2009, the discharge amount next to the left abutment increased again. It reached 14 liters per second and, at this time it carried coarse material, including the injected asphalt during 1989–1990. The COGERH is conducting a new campaign to evaluate the best alternative treatments according to the previous experience and new potential approaches that are able be tested. The treatment is taking place from February to June 2010, with simultaneous monitoring along the rain season.

In the spillway also occurred strong erosion all over the structures, starting with the approach channel and continuing under the control structure, till the restitution channel. The erosion on sedimentary rock resulted in many caves due to dissolution of interbeds cementation, some of them reaching deeper levels.

2.4 *Remedial campaigns*

The monitoring politicy carry out by Water Resources Secretary and COGERH was determinant to the dam safety, once the repairs took place when the anomalies were detected, even if they were performed in emergencial character.

From October 1989 to February 1990 it was executed a partial sealing grouting on the left abutment with a mixture of cement and asphalt. In the occasion it was registered a natural reduction of the discharge of 22 *l*/s, before the treatment had started. This treatment consumed 74,700 kg of asphalt, 1,118 kg of cement and resulted in a reduction of the discharge of 2.4 *l*/s. The attempt in using conventional grouting with cement was inefficient. All the material was carried down and emerged a few meters downstream the cross section in treatment. A better result was reached when using emulsion of asphalt in a preliminary stage of grouting, followed by cement. After that the seepage was steady with the discharge reduced from 22 to 2 *l*/s. Complementary repairs occurred also on the crest of the dam, with its reconstitution due to previous recorded settlement.

In the period of February and October, 1993, a campaign of treatment was started in the dam foundation, in it were drilled 3,303 meters and 329,973 kg of cement were injected.

From February 2002 to January 2003 some remedial services were executed in emergencial character, consisting of: reinforcement of the treatment of the dam foundation; replacement of the upstream riprap; construction of the flow gauges (Fig. 4). The draining system was modified to a rock-fill toe, due to the collapse of the previous conventional vertical and horizontal filters. A rock-fill toe was constructed with maximum height equal to 14 meters, at about 1/3 of the maximum hydraulic head (Fig. 3b); evaluation of the dam safety and; uprise of the crest level. In the dam foundation were drilled about 2,399 meters, and 120,750 kg of cement and 7,858 kg of bentonite were injected.

At the present a new campaign for sealing the region next to the left abutment is in progress. As pointed out before, the need for repairs occurred according to the demands that were detected throughout the monitoring process, as it can be seen on Table 1.

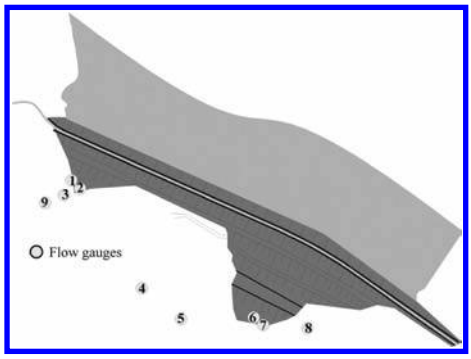

Figure 4. General view of the dam with the flow gauges.

Table 1. Resume of monitoring and repairements on Jaburu I dam.

Period	Event
1983	End of construction
1988	First fulfillment; Seepage on left abutment
1989–1990	Treatment on left abutment with cement and asphalt
1992	Seepage on the region next to outlet works and central valley
1993	Treatment of dam foundation
1997	Seepage above rock toe
2002–2003	Reinforcement of rock toe
2009	Seepage on the left abutment carrying out material
2010	Treatment on left abutment

3 DISCUSSION

Other experiences all over the word show that the current monitoring politicy in Jaburu I Dam is the

way to provide the dam safety despite its seepage anomalies.

The sandstones are usually associated to non conventional foundation support due to the causes briefly presented on Table 2. The strength depends upon the strength of the grains and of the cements or matrices and so the analyse of these aspects is very important on diagnosis of eventual incidents (Fell at al., 2005).

In Jaburu I Dam with the rock foundation dominated by porous type sandstones, the main effect of chemical weathering is the high conductivity due to removal of the cement matrix, visible in the flow gauges spread downstream the dam (Figs. 4, 5). The layers of siltstones that occur in the sandstone rock of the foundation can be the main cause for the developments of seepages. A secondary cause can be the intense fractures of the sandstone rock in the site (Fig. 6). Therefore, the differential settlement occurred on foundation, according to the hypothesis put above, might have caused formation of layers into the embankment and, in this order, collapse of the vertical drain.

Despite the recorded seepage all over the downstream of dam, the foundation has been treated

Table 2. Foundation characteristics and probable consequences.

Characteristics	Consequences
High porosity	Relative conductivity environments
Presence of gypsum as cement	High conductivity due to cavity formations
Interbeds of quartzite into its matrix	Variability of strength and variability of settlement
Interbeds of claystone into its matrix	Variability of strength and variability of settlement
Open joints and bedding surface crushed due to the stress relief	High conductivity and variability of settlement

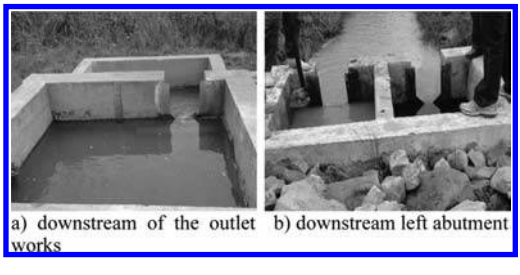

Figure 5. Sediment transported into seepage from the outlet works and the left abutment.

Figure 6. Typical aspect of the sandstone rock in the dam site.

in various stages, since its construction up to the present days. During the construction took place the conventional grouting campaign with cement, controlled by drilling water loss tests. The left abutment had not been contemplated with the grouting campaign with cement during the construction and it is probably that its interbeds rocks or cement matrix (Fig. 6) did not resist to the water pressure. Thus a preferential flow was established. In 1988, five years after the reservoir was completely filled up, and the discharge on left abutment reached the maximum recorded value of 47 *l*/s, no significant seepage was observed immediately along the downstream of the dam. This might mean that the conductivity in the foundation of dam was still under control and the water sought for the weakest environment next to the higher hydraulics head, the left abutment (Fig. 8).

According to Ceará (2010), the intervention during the period 1989–1990 should have been complemented with a grouting campaign with cement to allow the consolidation after the preliminary campaign with asphalt and cement, which reduced the discharge in about 90% (22 to 2,4 liters/second). Although the new campaigns in 1993 and 2002, when the reservoir reached again its maximum water level in 2009, the discharge increased again to critical ranges.

In Tim Ford Dam a similar anomaly was treated in a similar way (Bruce et al., 1998). Over there, the seepage problem was detected in a reservoir border next to the dam (left abutment), and the dam is only 13 meters higher than Jaburu I. The dam geotechnical site is described as a limestone sedimentary formation and the seepage reached 500 *l*/s, which was sealed with a treatment consisted of grouting cement with asphalt and calcium.

The case of Twin Buttes Dam can also be pointed out as a similar anomaly, where the seepage was controlled after the execution of protective impermeable walls with soil-cement backfill to assure adequate toe into low permeability bedrock (Dinneen & Sheskier, 1997).

Since 1993 seepages have being monitored through nine flow gauges, distributed throughout the downstream of the dam, according to the records presented in Fig. 7.

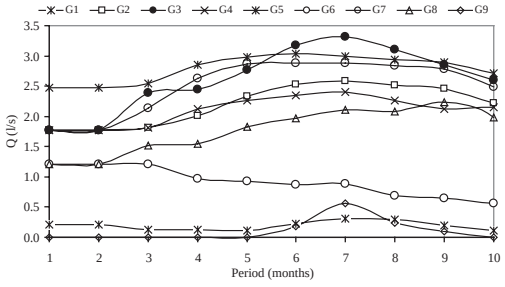

Figure 7. Measurement discharge (liters/second).

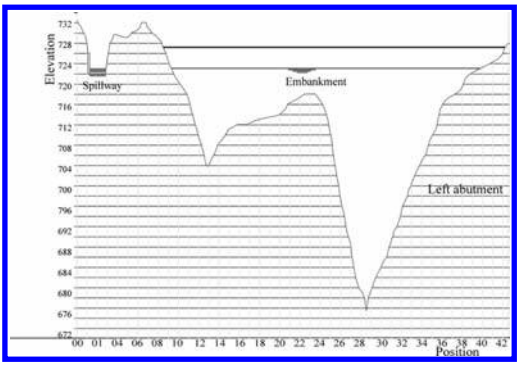

Figure 8. Section along the dam axis.

4 CONCLUSION

There is no doubt that if it was not the supervision and periodic inspections of dams and reservoirs their security would not be guaranteed. The periodicity is important because some anomalies cannot be detected early as the case of cracks into the embankment and only the consequences in form of seepage, if the structure is not provided by a reliable instrumentation system. The sequence of inspections report can be used to organize a classified system of intervention according to the risk magnitude. It is necessary for systems which are not provided with a robust financial support such as the one used in northeast region of Brazil. Sometimes the intervention happens in an emergencial character as it happened once in Jaburu I Dam. It must be pointed out that intervention in emergencial character is a bit better than intervention at the imminence of a structural collapse as sometimes it is possible to witness during the rainy season. In the case of Jaburu I dam it is a 100% positive that if it was not the controlling and remedial campaigns the structure would collapsed.

Brazilian committees of dams are involved in constant battles to aim a legal tool to control the numerous existing dams all over the country and it has produced some good results as the "law of dam safety" is currently being processed at the Federal Chamber.

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Safety and maintenance issues for a long embankment on a karstic foundation under artesian pressure

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ABSTRACT: The Riga run-of-river power plant in Latvia is located 20 km from the capital city of Riga in a rather flat area. The 380 m long powerhouse/spillway complex, designed for a head of 18 m, is flanked by 7.9 km and 7.0 km long homogeneous embankment sections on its right and left side, respectively. These embankment sections are founded on glacial, fluvio-glacial and alluvial deposits of variable thickness (3 to 10 m). They are underlain by a marly stratum which, however is not persistent, followed by a 6 to 12 m thick dolomite layer with solution features. The dam has performed well over a period of operation of more then 30 years. Of particular interest at this dam site is the foundation drainage which is affected by the karstic dolomite layer. The drainage water is collected in a drainage channel which has been rehabilitated in very recent time. It brings the water to the downstream area. The dolomite layer is partly fed by the reservoir creating an artesian condition with respect to the drainage channel. Pressure control is accomplished by relief wells, but still in some sections of the channel water from the dolomite flows upwards into the channel slopes, which can be observed as wet spots on the slope surface or as sand boils along joints in the channel lining. Of concern is also the flow regime in the dolomite layer which may lead to piping and to further dissolution of the rock. Rehabilitation and maintenance concepts for the drainage channel are briefly discussed.

1 INTRODUCTION

The Riga hydro-electric Power plant with a capacity of 402 MW is the lowest and most recent facility of the three power plants forming a cascade along the Daugava River in Latvia. It is located about 20 km from the capital city of Riga and about 35 km from the mouth of the river. The project was designed by Institute Hydroproject Moscow and is now owned and operated by AS Latvenergo. Construction started in 1966 and full operation with six units was achieved in 1976.

The layout of the facility consists of a more or less U-shaped, 15.4 km long, embankment dam with a maximum height of 25 m, a 220 m long powerhouse and a 160 m long spillway with six bays, split into two blocks (Fig. 1). The maximum discharge capacity of the concrete structures (spillway + six Kaplan turbines) is 12,300 m^3/s. The embankment is composed of five sections, namely (1) right bank embankment dam, (2) main dam no. 1, (3) island dam, (4) main dam no.2 and (5) left bank embankment dam (Fig. 2). The central part of the dam is founded on the former Dole Island which divides the river into two arms. The reservoir has a capacity of 256 Mm^3 at a normal reservoir level of +18 m asl.

The embankment was constructed by hydraulically placed fill of mainly glacial deposits (silty and clayey sand). The crest elevation is at +22 m asl. The upstream face is lined with concrete slabs as a protection from erosion by water waves, and the downstream slopes are covered with grass. Seepage control is by a drainage pipe inside the embankment and oriented in the direction of the dam axis, which brings the collected water to regularly spaced drainage collection wells from where it is discharged to a drainage channel. This channel also contains sections with relief wells which relieve the pressure from water circulating in partially and fully confined dolomite rock. The channel was rehabilitated in the years 2007/2008 (Fig. 11a and Figs. 12a and b).

Because of the proximity to the city of Riga, the safety of the dam is of foremost importance and dam and appurtenances are subject to rigorous safety inspections carried out periodically. No serious problems have occurred at this dam site since commissioning, however, repairs and other corrective measures have to be carried out within the

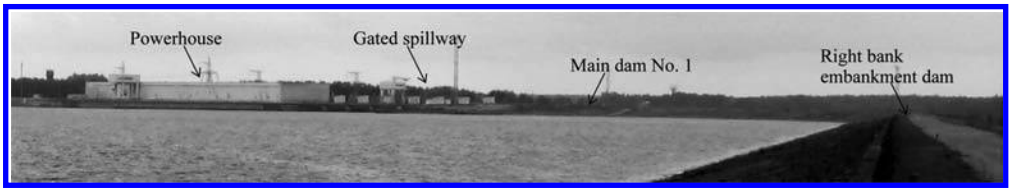

Figure 1. Powerplant: View from upstream right bank.

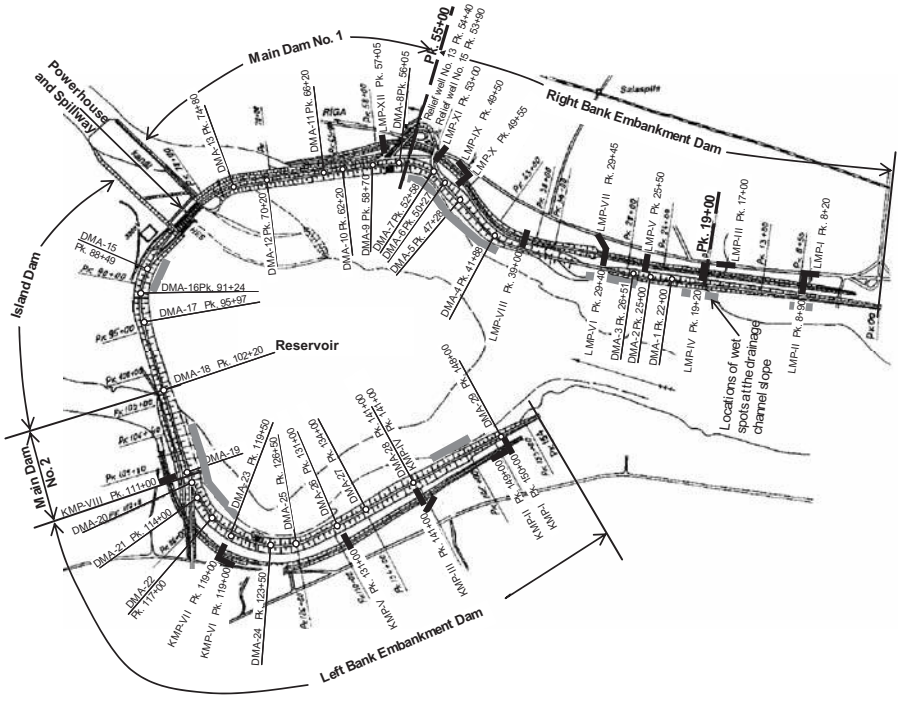

Figure 2. Layout of the Riga hydropower plant.

scope of the general maintenance and dam safety strategy set up for the facility. This paper points out problems that may compromise dam safety and how these are or will be handled by a rigorous maintenance and rehabilitation program, but it also demonstrates that rehabilitation work can lead to new problems.

2 GEOLOGICAL AND FOUNDATION CONDITIONS

The Riga powerplant is situated in the approximately 2.5–3 km wide Lower Daugava Valley, a rather flat area covered by Quaternary deposits (glacial, fluvio-glacial and alluvial), shaped by the last glacial stage and subsequently carved by Holocene erosion processes which finally led to the river valley with its terraces. The Quaternary deposits, 10 to 20 m thick, cover nearly horizontally bedded Paleozoic rock strata, the youngest belonging to the Upper Devonian era. Of relevance to the foundation and performance of the Riga dam are from top down: the Ogre clays and marls, the Daugava dolomite, the Salaspils marl and the Plavinas dolomite (Figs. 3 and 4). These rocks are underlain by the Amata-Gauja sandstones (in their top part often interstratified with marls), which form the most important resource for water supply in the Daugava Valley. The Amata-Gauja and also the Plavinas Formations are aquifers and the water they contain is under artesian pressure. These aquifers are alimented in the highlands of mainly Eastern Latvia. The Salaspils marl and also marly interlayers on top of the Amata series are considered impervious and represent a confinement for the aquifer. There may, however, be connections between aquifers, particularly in buried valleys,

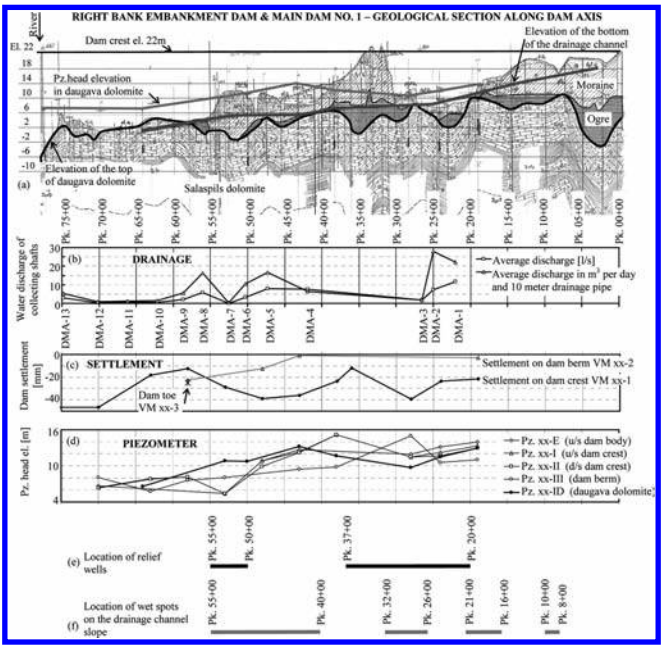

Figure 3. (a) Geological section along the dam axis of main dam No. 1 and right bank embankment dam; (b) Average water discharge since installation of drainage collection wells DMA-1 to DMA-13; (c) Dam settlement from construction until April 2008; (d) Piezometric head elevation in the dam and the foundation in 2008; (e) Location of relief wells in the drainage channel; (f) Location of wet spots on the drainage channel slope.

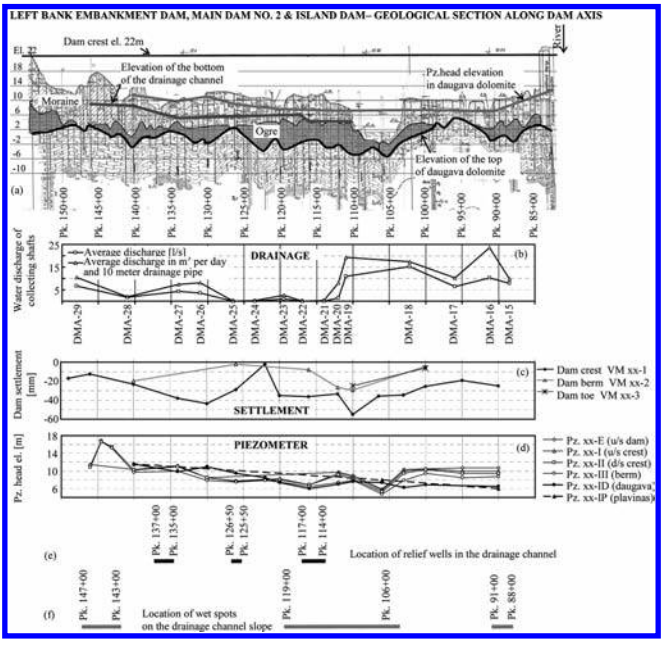

Figure 4. (a) Geological section along the dam axis of left bank embankment dam, main dam No. 2 and island dam; (b) Average water discharge since installation of drainage collection wells DMA-15 to DMA-29 (Note: record of DMA-14 is not available); (c) Dam settlement from construction until April 2008; (d) Piezometric head elevation in the dam and the foundation in 2008; (e) Location of relief wells in the drainage channel; (f) Location of wet spots on the drainage channel slope.

through pervious flow paths usually present in the valley fill.

The Daugava Formation is confined by the clayey Ogre formation, which, however, is not persistent and there are sections where the dolomite is in contact with the till/moraine or even with the alluvial deposits or the hydraulic fill of the dam. Therefore, piezometric heads in the Daugava dolomite will depend to some extent on the permeability of the contact material at the top of the dolomite stratum. The Daugava dolomite has a thickness of about 6–12 m.

The Ogre formation can be found mostly in depressions of the surface of the underlying Daugava Formation. Its thickness is between about 0.3 and 11 m and composed mainly of hard clay.

The Daugava formation is composed mainly of dolomites and dolomitic marls of greyish or yellowish-grey colour. Dolomite marls are stratified and sometimes platy. Clay interbeds can be found in both dolomites and marls. A special feature is the karstification of the Daugava dolomite. As a result of tectonic processes in the region, this very old carbonate rock is severely fractured, an ideal prerequisite for the development of karst. Along joints and especially at intersections of fractures, cavities and flow channels have developed. These solution features are mostly filled with sand and rock flour or calcite.

It is believed that the Daugava dolomite is fed mainly by the reservoir water at locations where the dolomite outcrops or is covered only by a thin and relatively pervious alluvial layer.

This is supported by temperature measurements in the relief wells which correlate well with the reservoir temperature. Due to its karstification, the permeability of the dolomite should be high. Rogal et al. (1999) report that during construction of the dam several sinkholes were encountered which caused certain difficulties. Chemical analysis of the reservoir and the drainage water from relief well No. 13 and 15 (Figs. 2 & 5) confirm that the seepage water may dissolve some magnesium from the dolomite layers. Estimating roughly the quantity, e.g. for relief well No. 15, with an average discharge of 50 l/s (Fig. 8b) shows that when the drainage water contains on average 5 ppm magnesium per liter (Fig. 5b), the loss per year is 7.9 tons or about 2.9 m^3. Models for the solution of limestone in karst are presented e.g. by Dreybrodt et al. (2002). They estimate the time until breakthrough of the karst front below a dam foundation. For limestone this time span can be 20 to 35 years depending on the hydraulic gradient, the solution capacity of the seepage water and the length of the seepage path. The solution rate of dolomite is somewhat smaller compared to limestone. In general, a reduction of the hydraulic gradient can result in a reduction of the seepage flow and water discharge which will slow down the karstification process.

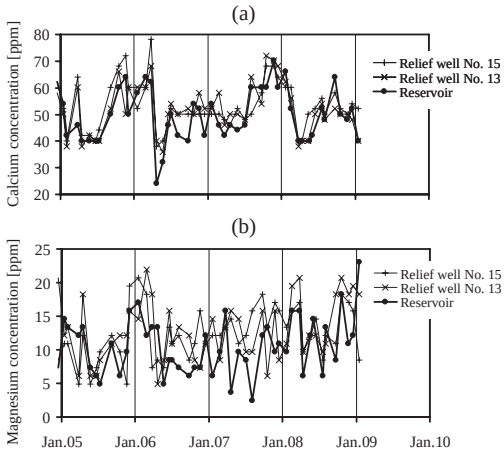

Figure 5. Chemical water analysis: Concentration of (a) Calcium; (b) Magnesium in the seepage and reservoir water.

3 DAM PERFORMANCE

The embankment dam is founded mainly on Quaternary deposits (alluvial sand, silty sand, fluvio-glacial gravel and pebbles, glacial deposits, etc.), but also on Ogre clay or Daugava dolomite or marl. Powerhouse and spillway are founded on a thin stratum of Salaspils marl with Plavinas dolomite below.

The embankment and the concrete structures are well equipped with the basic instrumentation, i.e. settlement points, standpipe piezometers and facilities (V-notch weirs) for measuring the seepage flow in the drainage collection wells. The embankment dam has 35 sections where settlements and pore water pressures can be monitored. A typical section is shown in Figure 6. Pore water pressure measurements start at Section 3 while settlements are surveyed starting from Station 1. Most sections have 3 to 5 piezometers and even more settlement points.

3.1 *Settlements*

There are more than 200 settlement points (VM), however, not all of these were installed at the same time. Settlement points are on the upstream side of the crest, on the berm and at the toe of the dam. The total settlements on the upstream side of the dam crest vary between 2.5 and 55 mm (year 2008, Figs. 3c and 4c). The differences in the settlement values are explained by the variable foundation conditions, the variable height of the fill, and by

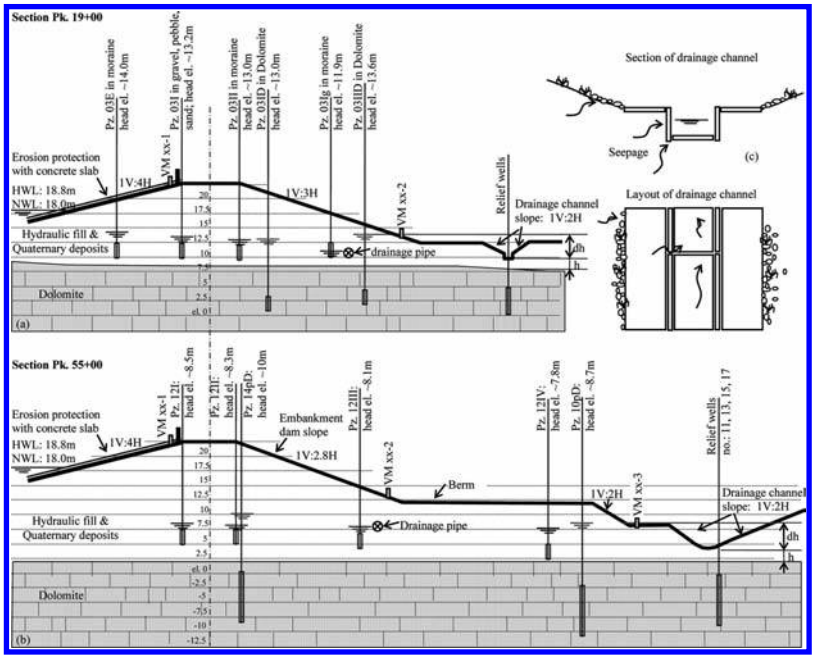

Figure 6. Cross-section of the embankment dam at (a) Pk. 19 + 00; (b) Pk. 55 + 00; (c) Section and layout of the drainage channel.

non-uniformities in the placement of the fill during construction.

There is limited information on settlements along the toe of the dam. These are considerably smaller than the crest settlements, except for the section between PK 58 and PK 62 where the toe settlements exceed the crest settlements because the settlement points were disturbed during road construction work.

3.2 *Pore water pressures*

Piezometric head elevations as recorded by piezometers in the dam fill and in the Quaternary deposit typically indicate values between about 5 m and 15 m with the highest values occurring along an approximately 2.5 km long section of the right bank embankment dam (PK 18 to PK 42), see Figs. 3d and 4d. In every section the piezometric head elevation decreases across the dam from the upstream to the downstream side.

Piezometers installed in the Daugava dolomite with the bottom of their approximately 2 m long filter screen at elevations ranging from −12 m to +2 m, give piezometric head elevations between about 6.0 m and 13.5 m asl. There are also some piezometers installed in the Plavinas formation with the bottom of their filter at about elevations −25 m to −33 m asl. and head elevations between 3.0 m and 11.5 m asl. The head elevations of the Daugava piezometers differ significantly from those of the piezometers in the embankment fill material (Figs. 3d and 4d).

The head elevations in the Daugava Formation exceed those in the fill material in the section from Pk 42 to Pk 58 by a maximum value of about 5 m. The reason for this behaviour is believed to be the variable materials at the contact with the dolomite rock. The pressure in the dolomite is increased where the rock is confined on top by a low permeability layer, such as the Ogre clay, whereas contact with more pervious Quaternary materials will allow some dissipation of the pressure.

However, where the piezometric head elevation in the Quaternary deposits or in the dam fill exceeds the one in the Daugava dolomite, water will still flow from the reservoir downward into the dolomite from where it flows in the more pervious dolomite layer to the drainage channel. This flow pattern has been modeled by the computer code SEEP/W which is part of the software package GeoStudio (2007). The model parameters were: Hydraulic conductivity in the dam fill, $k = 10^{-5}$ m/s, in the dolomite, $k = 10^{-4}$ m/s, concrete slab on upstream face, $k = 10^{-10}$ m/s, and drainage pipe as a drainage boundary. The results show clearly that the water enters the dolomite from the reservoir and exits at the relief wells in the drainage

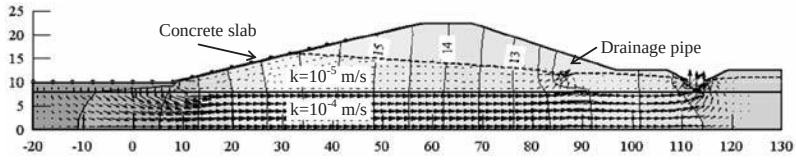

Figure 7. Seepage simulation at section Pk 19 + 00.

channel. In the embankment the flow is towards the drainage pipe (Fig. 7).

3.3 *Drainage facilities and volume*

The drainage pipe (Fig. 6) collects the seepage water in the dam material and guarantees the stability of the d/s embankment slope above the berm. The seepage water collected in the drainage collection wells DMA 1 to 29, (Figs. 3b and 4b) is measured before it is discharged to the drainage channel, usually by a V-notch weir.

The discharge in the drainage channel is measured, usually monthly, at several points, denoted as LMP-X and KMP-X along the right and the left bank respectively. Data are plotted for the period 1978 to 2008 for the right bank in Fig. 8a and there seems to be a trend of diminishing discharge. However, the accuracy of the measurement is considered rather low and the decrease does not necessarily mean a reduction in seepage. In some sections of the drainage channel, see Figs. 3e and 4e, there are free-flowing relief wells draining the Daugava aquifer. The contribution of these wells is also included in the discharge data. The drainage channel also collects water from small tributaries and of course also surface run off from rainstorms (Figs. 8b to d).

More reliable and accurate is the discharge measured in the drainage collection wells (DMA-xx, see Fig. 2 for their location). Figure 9 presents the discharge of DMA-1 to 4 installed along the right bank for the period January 2000 to 2009. The average discharge has remained more or less constant during the last 10 years. The data show significant variations, however, a seasonal pattern cannot be traced from the limited number of data per year. Still, in most of the DMAs the peak discharge occurs during autumn probably as a result of increased rainfall. There can also be other reasons for this seasonal pattern, however, the reservoir level is more or less constant with a drop of ~2 m during 2 weeks in summer for dam inspection (Fig. 10).

The average discharge in the drainage collection wells (Figs. 3b and Fig. 4b) depends on the length of the drainage pipe section, hence the average discharge in cubic meters (m^3) per day and 10 meter drainage pipe has been calculated and shows that the highest

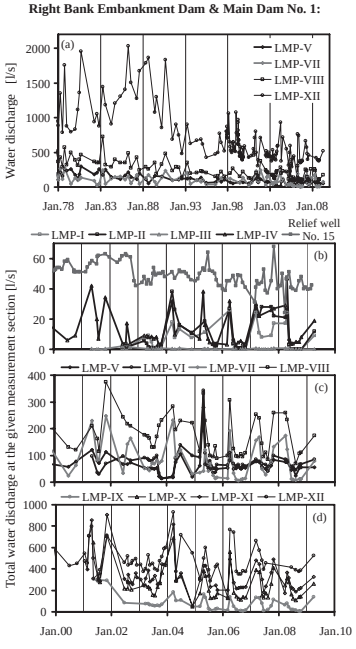

Figure 8. Water discharge in the drainage channel (a) since construction; (b) to (d) for the period January 2000 to 2009. Note: grey lines indicate side flows and black lines indicate drainage channel discharge.

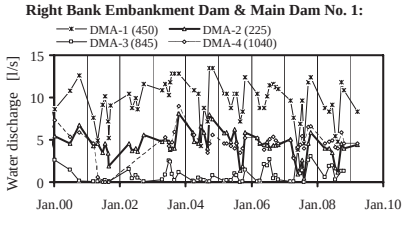

Figure 9. Discharge of collection shaft DMA-1 to DMA-13 for the period January 2000 to 2009.

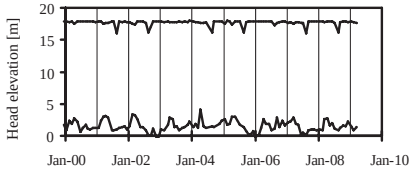

Figure 10. Reservoir and tailwater level.

discharge on the right bank is at Pk. 20 to Pk. 25 and at Pk. 40 to Pk. 60 which corresponds approximately to the locations of wet spots (Fig. 3f). On the left bank the highest discharge is between Pk. 90 and Pk. 110 which is the area of the main dam No. 2 and the island dam. In this area there is no drainage channel but a swampy area instead.

4 POSSIBLE FAILURE SCENARIOS

Considering the rather difficult foundation conditions, the embankment dam has performed surprisingly well. There are no signs or trends of critical developments. The solution features in the Daugava dolomite must, however, be considered as a persisting risk that can lead to piping through the embankment as sketched in Fig. 13.

There are two scenarios which must be considered in the safety assessment of the dam, namely (1) the integrity of the embankment vis-à-vis the possibility of initiating a piping incident at a dolomite/sand boundary and, in view of the upward acting gradient in the channel (Fig. 11, Fig. 12c and Fig. 14), (2) the stability of the drainage channel with respect to slope failure and to heave.

4.1 *Piping*

The process of piping in the case of a karstic foundation is somewhat different from the usual case where particles are transported through the voids of a coarser-grained material. Impounding of the reservoir creates a hydraulic gradient between the reservoir and the rock with solution features in the foundation. Impounding also increases the pore water pressure in the foundation rock where solution features are initially filled partly with soil material. Since the overburden is carried by the rock skeleton, the material in the solution feature is under a very low confining pressure.

Therefore the effective stress in the soil is low and the material can be eroded very easily by the water flowing through the solution features, even at small flow gradients. Hence the dolomite with soil-filled solution features can turn into a network of open voids (Schaefer, 2009). The initiation point of a pipe would be located where a solution feature is in contact with sand from the embankment or with quaternary deposits. From this point, backward erosion could start by washing material into the void which then is transported through a net of channels to a relief well. In the embankment the pipe would move upwards

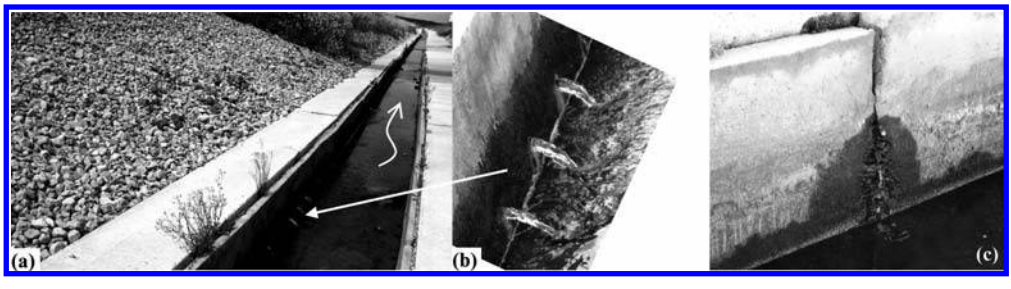

Figure 11. (a) Drainage channel; (b) Seepage from the left side through holes in the channel wall; (c) Seepage between joints of the channel lining.

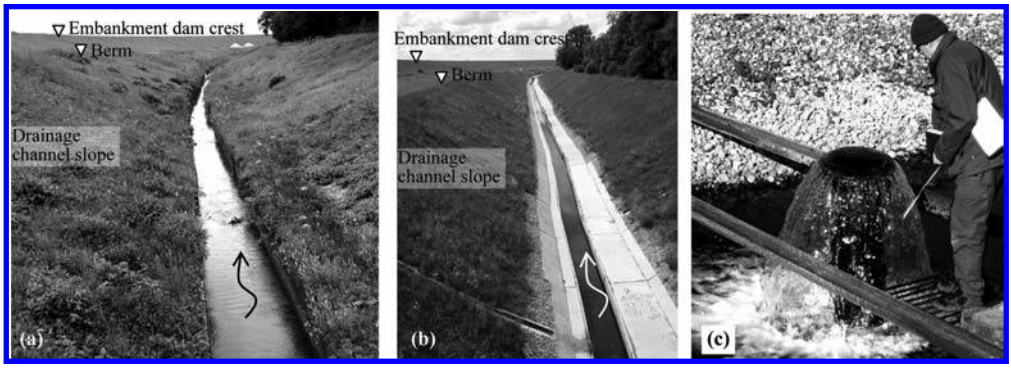

Figure 12. Overview of the drainage channel (a) before; (b) after rehabilitation; (c) Relief well No. 15.

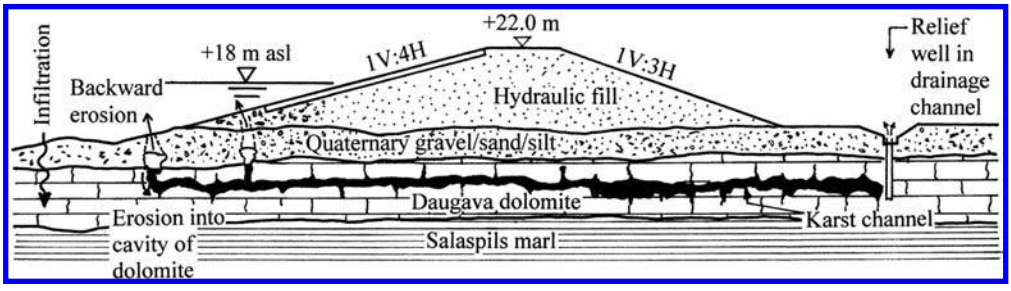

Figure 13. Erosion into solution features where dolomite is in contact with erodible material.

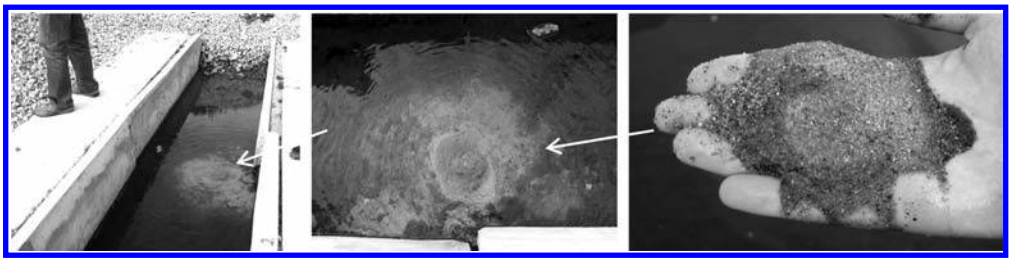

Figure 14. Sand boiling at Pk. 88 + 00.

until it reaches the reservoir. The pipe may also collapse and re-form taking a somewhat different path to the reservoir.

4.2 *Slope instability*

As a first approach the embankment dam slopes can be modeled as infinite slopes. Above the berm the dam has usually slopes of 1V:2.5 ÷ 3H. With an angle of friction for the hydraulically filled sand, estimated on the basis of its granulometry to be in the range of 30° to 33° (Bolton, 1986), the stability is assured when the drainage pipe at the dam toe is functioning properly. The drainage channel has usually slopes of 1V:2H. Their stability is not guaranteed when the slope becomes fully saturated, for example, as a result of upwards seepage from the artesian Daugava dolomite. Local channel slope failures are possible, however, it must be pointed out here that a failure in the drainage channel slope would in no case compromise the stability of the embankment dam.

The original concept of the drainage channel probably was to provide a means during embankment construction to dewater the site from the water draining from the hydraulic fill when it is consolidated. The relief wells served to drain pressured water from the Daugava dolomite, which may have caused some heave during construction but they were thought to be no longer necessary after the berm above the channel had been built.

4.3 *Channel base heave*

Modeling the drainage channel base as a block of soil with side friction neglected, shows that the block will not move when its height is larger than the head of the excess pore water pressure, assuming a unit weight for the soil of 20 kN/m^3. Numerous sections do not satisfy this condition, especially along sections where the dolomite is not overlain by the Ogre clay. Channel sections where the dolomite is covered by predominantly cohesionless Quaternary deposits may experience sand boiling with accompanied loosening of the layer as it can also be observed at some locations in the channel (Fig. 14).

5 REHABILITATION AND MAINTENANCE

5.1 *Rehabilitation design concepts*

The most important rehabilitation work carried out in recent time concerns the drainage channel (Figs. 12a and b). It allows for improved accessibility, detailed inspections of the channel linings, and intensified monitoring activities. Most of the relief wells date back to construction time, but they were buried, forgotten and finally re-discovered during the rehabilitation work. Some of them are damaged, filled with stones or are closed at the top with a steel plate. It seems that the contractor at that

time did not think these wells would be necessary for the operation or safety of the dam. Not all of the wells are discharging and among those which do there are large differences in the rate of discharge. Future rehabilitation work will provide the more important ones with electronic sensors for discharge measurement.

The special hydro-geologic situation at the Riga dam site provokes some thoughts about possible alternatives to handle the drainage associated with karst.

1. In the present concept drainage of the foundation seepage and uplift pressures are controlled by means of relief wells. Without these facilities the sliding stability of the channel slope and the uplift stability of the channel base would be critical. The relatively deep drainage channel collects the water from the relief wells, from the drainage collection wells and from tributaries. The water is then discharged downstream of the power plant at well defined points. Flow measurements in the channel are not precise and of low frequency, and it is not possible to detect whether the discharge of a particular relief well has changed. Therefore, it is also not possible to detect enlargements of the solution features in the Daugava dolomite along which the water flows to the relief wells. However, it is possible to monitor the content of solids in the drainage water from the relief wells.
2. In a second concept the idea is to reduce the flow velocity in the karst by reducing the hydraulic gradient. This could be accomplished by placing a geotextile on the Quaternary materials at the base of the channel and then backfill the channel partly with gravel derived from hard rock. Embedded in the backfill is a drainage pipe of sufficient capacity, which would collect the water from the drainage collection wells and from small tributaries. Due to the increased resistance to flow in the channel the piezometric head would increase in the dolomite and the hydraulic gradient decrease. This would also decrease the erosive forces and improve the safety against piping. The piezometric head elevation below the berm, however, should not exceed the elevation of the berm. The backfilled channel would end where it enters the flat area close to the river downstream of the plant. The difficulty in this concept is the proper monitoring of the drainage quantities.
3. A third concept would be the construction of a cutoff wall, e.g. a plastic concrete diaphragm wall, starting at the dam crest, passing through the Daugava dolomite and penetrating into the impervious Salaspils marl. This option has been employed frequently in the United States with dams on karst of up to about 2 km in length. For the Riga dam with a length of over 15 km this concept would make the project uneconomical.

5.2 *Maintenance strategies*

Maintenance work at the Riga dam site is intimately connected with dam safety issues, which in turn relate to the control of seepage and the proper functioning of the drainage system. That is why an open system with a drainage trench is preferable over a covered system with a buried drainage pipe, although the scope of maintenance is increased.

Improvements are also required in the discharge measurements of the channel flow to achieve a higher accuracy, say about 5%, such that a sudden increase in a particular LMP or KMP section could be detected and the location pinned down. Wet spots and locations with sand boiling must be protected with a filter in the form of a geotextile covered with sand and dolomite gravel, satisfying the appropriate filter criteria.

Monitoring seepage and drainage for safety surveillance shall be performed regularly, possibly always by the same group of people. Possible development of depressions on the dam crest, slopes or berms shall be viewed with great attention and provisions must be made for possible repair work. This also applies to the ground beneath the concrete slabs lining the upstream slope. The condition of the ground can be viewed during rehabilitation work of these slabs, which is still in progress.

6 CONCLUSIONS

Riga dam has performed well during its over 30 years of operation. Dam and appurtenances are maintained systematically and no serious incidents have occurred. Monitoring is performed consistently with relatively simple but mostly adequate instrumentation. The most important issue regarding dam safety is the control of seepage and drainage. The embankment dam has no cutoff trench and seepage control in the dam is accomplished by a drainage pipe and seepage collection wells while seepage in the foundation is controlled by a relatively deep-seated drainage channel, which recently has been rehabilitated. The latter has given rise to some concern because of the existence of a karstic dolomite stratum underlying the dam site with piezometric pressure head elevations exceeding the elevation of the channel.

Three concepts are discussed on how to handle optimally this particular hydro-geologic situation, namely: (1) the existing set-up at the dam site with an open channel and relief wells along

sections where certain phenomena, such as wet spots, sand boiling through the joints of the channel lining point to the existence of high pressures, can be observed. From numeric simulation, it appears that the karstic dolomite is mainly fed by the reservoir. There is also a certain risk for the development of piping, namely when cohesionless material from the layer above the dolomite would be transported into solution features of the dolomite. Rapid flow in a karstic dolomite channel leading to a relief well can be detrimental to the safety of the embankment as it promotes washing out of sediments from the solution features in the karst and it could further dissolve the rock. Hence, although the relief wells are helpful for stabilizing the channel, they also may contribute to intensify the solutioning of the rock and the erosion of soil deposits. (2) using a drainage channel lined with a geotextile, filled with gravel and drained by a pipe of sufficient capacity, which would receive the discharges from the drainage collection wells and possibly also from tributaries. The relief wells would discharge into this gravel pack but the flow quantity would be reduced. This would increase the pressure in the dolomite and lower the hydraulic gradient and thus the flow velocity for the transport of water in the channel. (3) the installation of a barrier cutting off the flow in the pervious strata, which would be required along the entire length of the embankment. In all three concepts the drainage pipe in the embankment is maintained as it has performed entirely satisfactory up to now.

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Las geomembranas sintéticas en la rehabilitación y mantenimiento de paramentos de presas

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RESUMEN: Los materiales orgánicos utilizados, comúnmente en España, en el mantenimiento de presas son los de aplicación "in situ"; mientras que las geomembranas sintéticas tienen, solamente, una presencia testimonial. A estos últimos pertenecen las barreras geosintéticas poliméricas empleadas en la protección de tres presas: Azud de Matavacas, Acanabre y Puente Porto. Los materiales utilizados son caucho butilo (IIR), poli(cloruro de vinilo) plastificado (PVC-P) y poliolefina termoplástica (TPO). Los resultados más significativos se presentan en este trabajo: espesores, características de tracción, doblado a bajas temperaturas, resistencia al impacto dinámico. resistencia al punzonamiento y microscopia óptica y electrónica de barrido. Asimismo, y en función de la naturaleza de la geomembrana, se han efectuado otras pruebas, como el contenido en plastificantes, resistencia al desgarro, dureza Shore-A y resistencia de la soldadura, tanto a tracción como a pelado.

ABSTRACT: In our country, the organic materials using in dam maintenance are liquid products. Synthetic geomembranes have only a testimonial presence. However, polymeric geosynthetic barriers are used in the protection of three dams: azud de Matavacas, Acanabre and Puente Porto.Butyl rubber (IIR), plasticized poly(vynil chloride) (PVC-P) and a thermoplastic polyolefin (TPO) are used as geomembranes. Results of evaluated characteristics are presented in this paper. These characteristics are thickness, tensile properties, dynamic impact, static puncture, low temperature folding, optic microscopy and electron microscopy "scanner". Also, and depending on the nature of the geomembrana, other test such as content of plasticizers, resistance to tear, Shore-A hardness and joint strength (shear and peeling test) are made.

1 INTRODUCCIÓN

En España, tradicionalmente, la rehabilitación de presas con materiales orgánicos se ha efectuado con productos poliméricos de aplicación "in situ". En cambio, el empleo de materiales macromoleculares en forma de geomembranas sintéticas es, casi simbólico, a pesar de que, en la actualidad, su utilización es bastante común en otros países.

La impermeabilización de presas con geomembranas sintéticas es un proceso, relativamente, reciente. Según datos del Boletín nº 78 del Comité Internacional de Grandes Presas (ICOLD 1991) traducido al castellano (Comité de Grandes Presas 1994) los materiales sintéticos que inicialmente se emplearon a este fin eran elastómeros y termoplásticos. Entre los primeros se empleó, fundamentalmente, caucho butilo. Presas en este contexto, cabe destacar la de Contrada-Sabetta en Italia-1959-, Miel en Francia—1968—y Sugarloaf en Australia-1976-. Como termoplásticos se hizo uso, principalmente, de poli(cloruro de vinilo) plastificado en las presas rumanas de Colibita y Mihoesti; las checas de Dosbina y Landstejn; la canadiense de Terzaghi; la francesa de Codole y la nigeriana de Jibiya. Otro material termoplástico mencionado es el polietileno de alta densidad empleado en la presa de Isanlu, también en Nigeria.

Si es importante el tema de la impermeabilización, no lo es menos la rehabilitación. Aparte de los clásicos métodos empleando materiales inorgánicos como en el azud de Estagento, se utilizan productos macromoleculares ya sean líquidos como en la presa de Cenza y El Tejo o en forma de geomembrana (Cazzuffi 1998, González 2002, Gonzalo et al. 2002, Martínez 2002, Scuero 1995, Blanco et al. 2005). A nivel internacional se ha hecho uso de poliisobutileno de 2 mm de espesor en una presa construida en el año 1930 que es la del Lago Baitone y cuya rehabilitación se realizó en el año 1971. No obstante, el material más empleado ha sido, de nuevo, el poli(cloruro de vinilo) plastificado utilizado al efecto en las presas de Heimbach, Lago Miller, Cignana, Lago Nero y Piano Barbellino.

En la actualidad, las geomembranas se siguen aplicando con profusión como en la americana

de Olivenhaim de 94.5 m de altura (año 2003), la de Balambano de 99.5 m en Indonesia (año 1999) y la colombiana de la Miel de 192 m (año 2002) (Scuero & Vaschetti 2002, Scuero 2004).

A finales de los años sesenta y en la década de los setenta, se comienza a emplear en nuestro país estos sistemas novedosos bien como membrana interna tal es el caso de la presa del Odiel-Perejil (Blanco & Zaragoza, 2003) o externa como las de Aiguamoix o el azud de Matavacas (Blanco et al. 2003, Cea et al. 2003, Leiro et al. 2002). Más recientemente, en la rehabilitación de la presa de Los Melonares, se ha hecho uso de una geomembrana de poli (cloruro de vinilo) plastificado. (Scuero et al. 2008).

2 MATERIALES SINTÉTICOS

2.1 *Productos líquidos*

En este grupo se incluyen materiales de distinta etiología, donde caben destacar productos como poliuretanos, resinas epoxi, resinas acrílicas, polisulfuros, entre otros.

A nivel europeo los materiales de aplicación "in situ" no están, actualmente, en proceso de normalización y sus recomendaciones aparecen en unas guías publicadas por la EOTA. En cambio, a escala nacional se han elaborado unas normas que cubren todo el campo de la obra hidráulica. En ellas se citan las características, la metodología experimental para determinarlas, su aplicación y los requerimientos mínimos exigibles. Dichas normas son las que se citan a continuación:

UNE 104 309-1 Impermeabilización. Materiales líquidos para la impermeabilización de conducciones empleadas en obras hidráulicas. Características, métodos de ensayo y puesta en obra.

UNE 104 309-2 Impermeabilización. Materiales líquidos para la impermeabilización de paramentos de presas. Características, métodos de ensayo y puesta en obra.

UNE 104 309-3 Impermeabilización. Materiales líquidos para la impermeabilización de depósitos empleados en obras hidráulicas. Características, métodos de ensayo y puesta en obra.

UNE 104 309-4 Materiales para la impermeabilización de juntas de trabajo y construcción en conducciones, presas y depósitos impermeabilizados con materiales líquidos. Características, métodos de ensayo y puesta en obra.

2.2 *Geomembranas*

La normativa europea agrupa a los materiales poliméricos para impermeabilización en tres clases según sus propiedades físicas (Tabla 1):

I. Termoplásticos
II. Cauchos termoplásticos
III. Elastómeros

Tabla 1. Naturaleza de las geomembranas sintéticas utilizadas en la impermeabilización de presas.

Nomenclatura	Material básico	Clase
ECB	Copolímeros de acrilatos/ etileno y betún	
EVA/C	Copolímeros de acetato de vinilo y etileno	
EEA	Copolímeros de acetato de etilo y etileno	
PE	Polietileno	I
PEC	Polietileno clorado	
PP	Polipropileno	
PIB	Poliisobutileno	
PVC-P	Poli (cloruro de vinilo) plastificado	
TPO	Poliolefina termoplástica	
E/P	Copolímeros de etileno/ propileno	II
CSM	Polietileno clorosulfonado	
CR	Caucho de cloropreno	
EPDM	Caucho terpolímero de etileno propileno-monómero diénico	
IIR	Caucho butilo	III
NBR	Caucho de acrilonitrilo/ butadieno	
BR	Caucho de butadieno	
POE	Poliolefina elastomérica	

La norma europea "UNE-EN 13361 Barreras geosintéticas. Características para su utilización en la construcción de embalses y presas" define las características así como los métodos de ensayo a llevar a cabo con objeto de conocer el comportamiento de las geomembranas utilizadas y, asegurar la estanquidad en obras hidráulicas; pero al igual que ocurre con otros geosintéticos (geotextiles, geodrenes, ...) no se indican unas exigencias en cuanto a valores; la norma europea, como tal, no considera cálculos ni requisitos de diseño o dimensionado, ni tampoco cubre las técnicas de instalación de dichas geomembranas.

Los requerimientos a exigir para cada material en función de su naturaleza deben buscarse en otras fuentes: normas de puesta en obra, reglamentos o manuales sobre el tema (MMARM 2010). Información exhaustiva sobre el empleo de geomembranas en presas está reflejada en el último documento de la "International Comission on Large Dams" (ICOLD, 2010).

3 CASOS PRÁCTICOS

3.1 *Azud de Matavacas*

El azud de Matavacas se encuentra situado en el municipio de Sanlúcar de Guadiana en la provincia de Huelva, a una cota de 122 m (Fig. 1). Es un

buen ejemplo de la instalación de una pantalla impermeable, en el paramento de aguas arriba de una presa de escollera. En este caso, se empleó una geomembrana de caucho-butilo (IIR) de 1 mm de espesor, que era el material sintético más utilizado en el año 1974, fecha de su puesta en obra (Cea et al. 2002). La capacidad del azud es de 0.3 hm^3 con una altura de 10.5 m.

Aunque la barrera geosintética polimérica se instaló en el año 1974, es desde hace diez años cuando se está llevando a cabo un seguimiento periódico sobre su comportamiento. Los resultados obtenidos al cabo de los treinta y tres años de su instalación son los que figuran en la Tabla 2.

En algunos casos las probetas de soldadura extraídas del propio azud, han roto por la propia unión entre paños, como consecuencia del tiempo transcurrido desde que se realizó dicha unión.

Figura 1. Azud de Matavacas (Sanlúcar de Guadiana. Huelva). La impermeabilización se realizó con caucho butilo.

Tabla 2. Características de la geomembrana de IIR a los 33 años de su instalación.

Característica	Unidad	Valor
Espesor	mm	1.02
Dureza shore-A	–	71
Doblado a bajas temperaturas (−40ºC)	–	Bien
Resistencia la impacto dinámico		
Altura de caída del percutor	mm	>300
Resistencia a la tracción	MPa	9.7
Alargamiento en rotura	%	226
Impacto estático		
Resistencia al punzonamiento	N/mm	301
Recorrido del punzón antes de perforar	mm	27
Resistencia de la soldadura por tracción	N/50 mm	220

Las muestras procedentes de la geomembrana del azud se han sometido a una prueba de microscopia óptica de reflexión a distintos aumentos y por ambas caras. No se ha podido realizar un estudio por microscopia electrónica debido a la interacción entre el negro de carbono de la superficie de la geomembrana y el propio microscopio. Las condiciones en que se han realizado las pruebas se encuentran ampliamente reflejadas en la bibliografía científica (Soriano et al. 2010). En la Figura 2 se puede observar que la cara exterior está más deteriorada como consecuencia de la incidencia de las radiaciones solares sobre la misma; la cara interna está en mejor estado de conservación. En cualquiera de los casos se puede comprobar que se trata de superficies homogéneas y uniformes.

3.2 *Presa de Acanabre*

Está ubicada en el término municipal de Alajeró, en la isla de La Gomera, próxima al Parque Nacional de Garajonay por lo cual los requerimientos medioambientales fueron notables (Rosa et al. 2008, Abad et al. 2010).

La antigua presa de hormigón de Acanabre, situada en el arroyo del mismo nombre, no conseguía almacenar agua por la alta permeabilidad del terreno. Las necesidades de agua para el riego, han obligado a ampliar su altura mediante un terraplén de recrecimiento que se apoya en el cuerpo de la presa. La galería de la presa se ha utilizado para el desagüe de fondo de la balsa.

La entrada de agua es doble y se realiza por coronación desde los arroyos existentes. En ambos casos se adopta idéntica configuración. Por encima se construyó con gaviones, una albarrada de 3.00 m de altura para contención de los elementos sólidos que pueden dañar la geomembrana. Asimismo, en la coronación se dispuso de dos badenes de 5.00 m de ancho para entrada del agua. Estos badenes dan continuidad al camino de coronación cuando no hay escorrentía.

La aparición de varios manantiales en las laderas, obligó durante la obra a realizar su captación y saneamiento mediante grava envuelta en geotextil, cuyo que desagüe a través de tubería se integra en la red de drenaje.

Se ha construido un nuevo aliviadero en la margen izquierda del cauce con una capacidad de 5.50 m^3/s que está constituido por un canal de 3.60 m de ancho y 1.50 m de altura que discurre desde la coronación al pie de la presa.

La presa de Acanabre se encuentra a una cota de 1258.5 m. Su capacidad es de 45,000 m^3, altura 8 m, pendiente de talud 3/1 y un perímetro de coronación de 370 m (Fig. 3). Se han utilizado 13,000 m^2 de una lámina de poli(cloruro de vinilo) plastificado de 1.5 mm de espesor, cuyas uniones entre paños se han realizado por vía térmica.

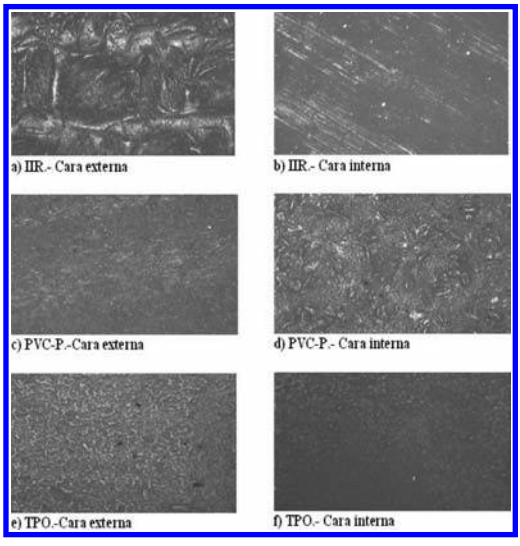

Figura 2. Microfotografías (× 60) de las muestras de geomembranas de IIR (Azud de Matavacas), PVC-P (Acanabre) y TPO (Puente Porto), por ambas caras, obtenidas al ser observadas por microcopia óptica de reflexión.

Figura 3. Presa de Acanabre (Alajeró.—Isla de La Gomera). Su rehabilitación se llevó a cabo con poli (cloruro de vinilo) plastificado.

El geotextil empleado fue polipropileno de 400 g/m^2 que se situó sobre una capa de hormigón poroso de 10.00 cm de espesor dispuesto sobre los taludes. El anclaje de la lámina se realizó mediante zanjas en coronación, lastres de hormigón en la base y pletina atornillada en las obras de fábrica.

Los resultados obtenidos para la geomembrana sintética de poli (cloruro de vinilo) plastificado son los que figuran en la Tabla 3. Además, y teniendo en cuenta la migración de plastificante que conlleva la degradación de la lámina, se ha determinado el contenido en este aditivo que resultó ser 31.8%.

Tabla 3. Características iniciales de las geomembranas sintéticas procedentes de las presas de Acanabre (PVC-P) y Puente Porto (TPO).

	Valores	
Características	PVC-P	TPO
Espesor, mm		
Nominal	1.52	2.54
En el punto de cruce del tejido	0.40	1.00
Doblado a bajas temperaturas	Bien	Bien
Resistencia al impacto dinámico	Bien	Bien
Resistencia a la tracción, N/50 mm		
Longitudinal	1450	1073
Transversal	1287	1110
Alargamiento en el punto de carga máxima, %		
Longitudinal	26	68
Transversal	27	80
Resistencia al punzonamiento, N/mm		
Cara externa	410	472
Cara interna	393	453
Recorrido del punzón antes de perforar, mm		
Cara externa	20	25
Cara interna	20	27

Se ha determinado, asimismo, la resistencia de la soldadura entre los distintos paños, tanto cualitativa, es decir por traccción, como cuantitativamente a pelado. Los valores medios alcanzados fueron:

Resistencia de la soldadura, N/50 mm	
Por tracción	1135
A pelado	393

En el caso de la soldadura por tracción la rotura tuvo lugar en el borde o en las proximidades de la unión, pero fuera de la soldadura propiamente dicha.

En la Figura 2 se presentan las microfotografías obtenidas al ser observada la muestra por Microscopia Óptica de Reflexión (MOR) y en la Figura 4 las relativas a su inspección por "scanner" o Microscopia Electrónica de Barrido (MEB). La tecnología MOR refleja un material en muy buen estado de conservación; sin embargo el "scanner" (MEB) detecta ciertas microfisuras en el material sintético.

3.3 *Presa de Puente Porto*

La presa de Puente Porto se encuentra ubicada en el municipio de Galerne en la provincia de Zamora a una cota de 1646.2 m (Fig. 5). Tiene una capacidad de 23 hm^3, una altura de 32 m y un perímetro de coronación de 325 m. Se utilizó para

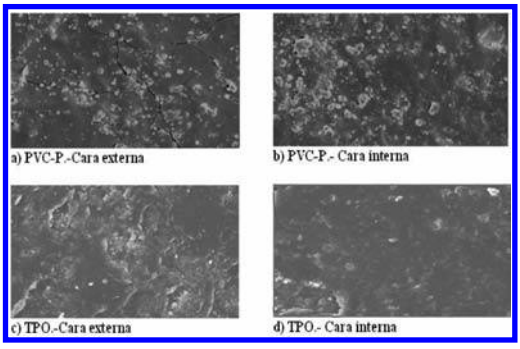

Figura 4. Microfotografías (× 900) de las muestras de geomembranas de PVC-P (Acanabre) y TPO (Puente Porto), por ambas caras, obtenidas al ser observadas por microcopia electrónica de barrido.

Figura 5. Presa de Puente Porto (Galerne. Zamora) impermeabilizada con una geomembrana sintética a base de una poliolefina termoplástica.

su impermeabilización una geomembrana sintética de poliolefina termoplástica (TPO) sobre un geotextil de 1000 g/m^2.

Los valores alcanzados se presentan en la Tabla 3 También se determinó la resistencia al desgarro, tanto en sentido longitudinal como transversal, obteniéndose un valor de 325 N en ambos casos.

En la Figura 2 se presentan las microfotografías obtenidas al ser observadas las muestras por Microscopia Óptica de Reflexión (MOR) y en la Figura 4 las relativas a su inspección por "scanner" o Microscopia Electrónica de Barrido (MEB). Ambos tipos de microscopia muestran un material homogéneo y uniforme así como ausencia de microporosidad y otros defectos superficiales.

4 SEGUIMIENTO

Desde el año 1989 el CEDEX viene realizando un control periódico a las geomembranas sintéticas utilizadas en obras hidráulicas; en estos momentos dichos controles abarcan a unas ciento cincuenta estructuras. (Aguiar & Blanco 1995, Blanco et al. 1998, Blanco et al. 1999, Blanco 2009). Al tratarse de ensayos destructivos, se pretende tomar la mínima cantidad de material para no deteriorar al sistema impermeabilizante, pero que sea suficiente para determinar con claridad su estado en el momento de la extracción. Las pruebas a realizar dependerán del tipo de material utilizado. Las más comunes, en obras hidráulicas, son las siguientes:

Espesores
Características de tracción
Doblado a bajas temperaturas
Resistencia mecánica a la percusión (Impacto dinámico)
Resistencia a la perforación (Impacto estático)
Contenido en plastificantes (PVC-P)
Resistencia al desgarro (PEAD)
Negro de carbono y su dispersión (PEAD)
Resistencia de la soldadura por tracción
Resistencia de la soldadura por pelado
Microscopia óptica
Microscopia electrónica de barrido

5 CONCLUSIONES

De los resultados obtenidos en las pruebas efectuadas en las geomembranas utilizadas en la protección de presas, a las que hace referencia este artículo, se podrían extraer las siguientes conclusiones:

– La geomembrana de caucho butilo instalada, hace más de treinta años, en el azud de Matavacas sigue conservando unas propiedades aceptables, exceptuando la pérdida de alargamiento en la rotura y la unión entre paños. La microscopia óptica de reflexión muestra un material homogéneo y uniforme por ambas caras, detectándose en la externa un cierto deterioro como consecuencia de la incidencia de las radiaciones solares sobre la misma.
– La muestra de poli (cloruro de vinilo) plastificado procedente de la presa de Acanabre, conduce a unos resultados acordes a los requerimientos exigidos a este tipo de producto. Cuando se efectuó la prueba de resistencia de la soldadura por el método de tracción, todas las probetas rompieron en las proximidades de la mencionada soldadura, pero siempre fuera de la unión propiamente dicha. Tanto la microscopia óptica como la electrónica de barrido indican un material con una cierta microporosidad y alguna microfisura.
– La poliolefina termoplástica, colocada como pantalla impermeable en la presa de Puente Porto, presenta una elevada carga de rotura debido al textil que lleva como refuerzo. Es de

destacar su excelente comportamiento, tanto en la resistencia al punzonamiento estático como en el doblado a bajas temperaturas. La microscopia óptica de reflexión al igual que el "scanner" o microscopia electrónica de barrido presentan un material en muy buen estado, no detectando ningún tipo de defecto superficial.

A la vista de los resultados obtenidos, en cada uno de los materiales utilizados en las presas mencionadas en este trabajo, sería conveniente afianzar estas conclusiones con más resultados a través de futuros controles periódicos.

Las causas individualizadas, para cada uno de estos materiales y presas, por las que sería de gran interés llevar a cabo dichos controles periódicos se exponen a continuación:

- La geomembrana de IIR del azud de Matavacas, debido al tiempo que lleva instalada precisa de un seguimiento que genere la información necesaria para analizar su comportamiento y, como consecuencia el momento de reposición de la pantalla impermeable, con objeto de garantizar la seguridad del embalse.
- El PVC-P de la presa de Acanabre, al ser plastificado, tiene tendencia a la pérdida de plastificantes, lo que conlleva la disminución de todas sus propiedades y por ello la degradación del material. Por este motivo, es esencial efectuar pruebas periódicas que pongan de manifiesto la pérdida del mencionado aditivo. Esta información permitiría conocer el momento en el que dicho material dejaría de cumplir la función para la que ha sido destinado.
- El TPO, por tratarse de un material de nueva implantación, debería ser objeto de un control periódico adecuado, ya que su comportamiento en el tiempo es desconocido.

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Project for reconstruction of *Marquesa* weir, river *Júcar*, township of *Cullera* (*Valencia*)

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ABSTRACT: The *Marquesa* weir is located in the *Júcar* river, approximately 4.5 km from its mouth in the Mediterranean Sea, in the township of *Cullera*. It has a length of 210 m and an average crest elevation of 1.75 m. Its function is twofold, environmental (it prevents upstream saline intrusion) and irrigation. The successive river floods have damaged the weir, which has been repaired by the addition of rip-rap and the replacement of the concrete shell. To the deterioration of the weir itself, we must add the bad state of the river banks from an ecological point of view and the fact that the dam poses a barrier to the fish fauna. The scope of the project is therefore to ensure compliance with the current functions of the dam through its reconstruction and to restore the river environment as well as fish transit.

1 LOCATION AND CURRENT SITUATION

1.1 *Introduction*

The irrigation system of *Cullera* derives most of its water from the weir of *Cullera*, located in the meander of *Fortaleny*, at an altitude of 8 m. Upstream the mentioned dam we find the intakes of the main irrigation ditches, called *Acequias Madre*, one in each *Júcar* bank and dating from the XVth and XVIth centuries. The right margin main ditch splits into three ditches when entering the municipality of *Cullera*, one of them irrigates the lands of the right riverbank and heads toward the *Marquesa* weir, that once gave service to a watermill on the left bank.

The river stretch between both dams acts as a regulation reservoir for a number of intakes which, with extraordinary character irrigate the surrounding land. It also receives the surplus water irrigation from the upstream network.

1.2 *Location*

The *Marquesa* weir is located in the *Júcar* river, approximately 4.5 km from its mouth in the Mediterranean Sea, in the township of Cullera.

It is included in the site of Community importance (SIC) *Curso Medio y Bajo del río Júcar*.

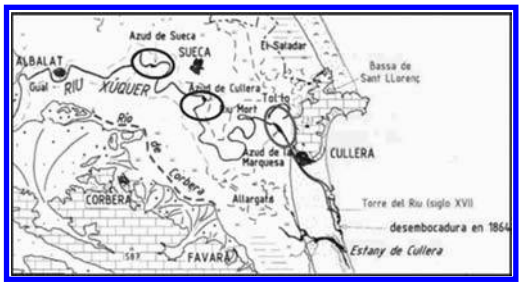

Figure 1. Location of dams in the district *Ribera Baja del Júcar*.

1.3 *Dam structure and current state*

The current dam structure is as follows:

- Foundation based on a grid of wooden piles with 15 cm diameter, forming a grid of 3 m side.
- Section constituted by riprap and graded aggregates.
- Surface of concrete shell with a thickness of 20 cm; the weir has a surface width of 10 to 15 m and a height of about 2 m. Toe shape: abrupt step.

It has a length of 210 m and an average crest elevation of 1.75 m.a.s.l.

The successive river floods have damaged the weir, which has been repaired by the addition of riprap and the replacement of the concrete shell.

1.4 *Dam functions*

Its function is twofold:

- Environmental: it prevents upstream saline intrusion in the *Júcar* river, thus contributing to the maintenance of the river ecosystem and enabling the use of water for irrigation.
- Irrigation: it facilitates the operation of irrigation intakes between *Marquesa* and *Cullera* weirs (dealt with by irrigation union groups in *Sueca* and *Cullera*), as well as the operation of the intake "Connection of the *Júcar* lower reach with stretch V. New *Júcar-Vinalopó* pipe" currently in execution. It guarantees the disposal of fresh water for all the intakes.

1.5 *Banks characterization*

From the point of view of its banks, the studied area is characterized by:

- being an area where natural vegetation has been transformed due to high anthropic pressure, where the main use is the agricultural one (mainly orange trees).
- The massive proliferation of reed (*Arundo donax*). Riparian trees species such as poplars (*Populus nigra* and *Populus alba*), elms (*Ulmus minor*) and willows (*Salix*) have virtually disappeared nowadays. Only a few specimens of *Populus alba* can be found.
- The erosion detected during the bathymetry in the river-bed and the banks both upstream and downstream of the dam.

Figure 2. Cracks and seepage in the dam crest.

1.6 *Longitudinal continuity: Fish transit*

At present, the *Marquesa* weir poses an insurmountable barrier to the fish fauna both for its dimensions and for the lack of a fish-pass. The Water Framework Directive (2000/60/EC) requires Member States to reach at least a 'good ecological and chemical status' in all their water bodies before 2015. Guaranteeing the rivers continuity is considered a contribution to good ecological status under this Directive and must be defined in terms of free movement of all the non-exotic fish species present in the stretch.

The autochthonous fish species present in the studied river reach are mainly the eel (*Anguilla anguilla*) and the barbel (*Barbus guiranois*).

1.7 *Analysis of the hydraulic heritage*

The *Marquesa* weir was designed to divert water to the *Marquesa* watermill, now in disuse, through the ditch of the same name.

Nevertheless its reconstruction is vital to fulfil its present roles. The works will include the demolition of the existing concrete shell and the partial elimination of the riprap placed in contemporary times. This action will be carried under archaeological control, allowing to study what the original dam was like; depending on its conservation status it could be acted upon in order to show the public how this hydraulic work was built.

The remaining patrimonial assets located next to the dam, the watermills of the *Marquesa*, *Cardona* and *Cremat* won't be affected by the works.

1.8 *Hydrological analysis*

For return periods exceeding 25 years the flow in the *Júcar* Plain is very complex. Limiting its analysis to the area lying between the highway A-7 and the sea, there are three preferred flow directions: overflow through the left margin towards *L'Albufera*, overflow through the right bank (of lower intensity than the previous one) towards the *Marjal* (Marsh) and flow through the river *Júcar*, as shown in next figure.

Moreover, because of the barrier effect exerted by the railway platform, water overflowing from different *Júcar* cross-sections accumulates west of it, the railway performing a task of retention and peak flow lamination, as shown in Figure 3.

Therefore the further hydraulic study has sought to evaluate the real present and future capacity of the river in the area of the *Marquesa* weir, using only as a reference the theoretical flow values obtained by hydrological methods for different return periods.

Stability analysis of the dam, which requires computation for higher return periods, has been carried

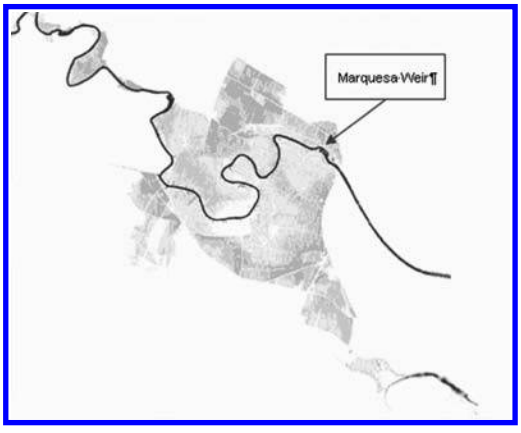

Figure 3. Water depth caused by overflows in the *Júcar* in the area of *Marquesa* weir for a return period of 25 years, *CEDEX*. We may clearly see the barrier effect created by the railway.

Table 1. Water Surface Elevation (WSE) upstream and downstream the *Marquesa* weir cross-section.

Return period T or flow Q (m^3/s)	Upstream WSE	Downstream WSE
Ordinary conditions**	1,75	0 (sea level)
Q = 355 m^3/s	2,50	2,42
T = 25	3,33*	3,06*
T = 100	3,54*	3,38*
T = 500	3,61*	3,46*

*: values according to CEDEX.
**: Ordinary conditions: ecological flow over the weir.

out using the values of water depth obtained by the *CEDEX,* whose two-dimensional model *GIS-PLANA* allows to reproduce the widespread overflows in the *Júcar* plain.

2 PURPOSE AND CONDITIONING OF THE DAM RECONSTRUCTION

We propose therefore the project with the following scopes:

– To ensure compliance with the current functions of the dam through its reconstruction. Imperviousness will be insured through the construction of cutoff walls.
– To restore the river environment in order to reach, as far as possible, its original condition not only in geomorphologic and functional aspects but also in those biological and landscape, with the main aim to recreate habitats that promote the diversity of both flora and fauna.
– To restore fish transit through a lateral fish passage.
– To increase its hydraulic capacity.

The actions proposed fall within the National Strategy for Restoring Rivers, as much for the weir acquired permeability to the fish fauna passage as for the restoration of the surrounding shores.

The main assumptions considered both during the execution of works and after them are:

– Not undermining the operating conditions of the existing water intakes upstream of the dam: the current weir crest elevation should be maintained. Therefore the crest elevation of the reconstructed dam should be greater or equal to the current one. Also, the proposed construction procedure must ensure maintenance of current water elevation.
– The separation between the downstream saline water and the upstream fresh water used for irrigation should be ensured: a barrier must continually be maintained between both water-bodies (using sheet piles during construction and the dam itself afterwards). The conductivity values in both water-bodies are given in the following figure.
– One of the scopes is to restore the biological continuity and to ensure the hydraulic one, allowing at all times the passing both of the ecological flow and of the fish native to the area, through the fish pass and the lateral diversion channel.
– The strict hydraulic capacity (the one obtained when the water surface elevation reaches the banks elevation, in this case 2.5 m, just before overflow) of the existing weir, equal to 220 m^3/s is not to be diminished.

During the works it is guaranteed in case of floods considering the capacity both of the diversion channel and of the river cross-section over the executed cofferdam piling.

After the works capacity is increased with the new design of the dam through the use of an USBR profile.

– One of the pursued scopes has been to minimize the environmental impact: namely minimizing the surface of the river affected by the works, the main consideration for the selection of the construction process. The balance of the earth movement has also been sought, by using the material existing in the parcels affected by the works and by avoiding borrowing for soil and topsoil fillings. Thus the only materials to be dumped will be the products of demolitions, plant material non-usable on site (soil with rhizomes, clearings, logging, etc.) and the surplus riprap. The waste management norm in force is complied with (*RD 105/2008*).

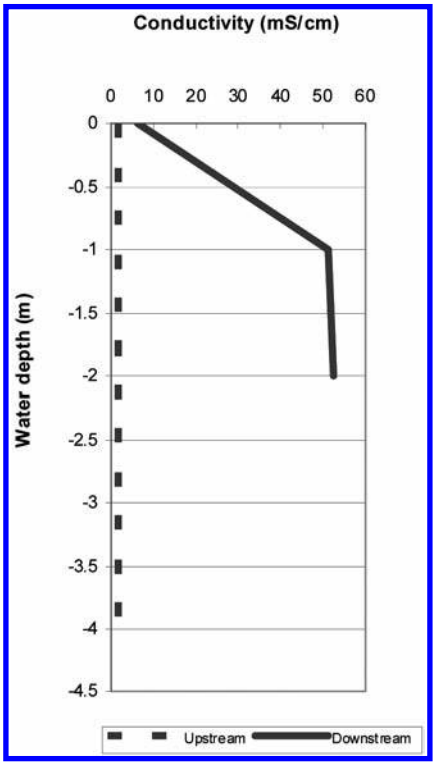

Figure 4. Values of measured conductivity upstream and downstream the *Marquesa* weir.

3 ANALYSIS OF ALTERNATIVES

3.1 *Hydraulic analysis*

Three alternatives were considered for the weir discharge profile:

- The first would be to maintain a hydraulic behaviour identical to the current one, proposing a new dam with a shape similar to the existing one (broad-crested weir, usptream edges without rounding).
- The other two analyzed alternatives would instead increase the hydraulic capacity of the current dam. Alternative 0 consists in a sharp-crested weir. Alternative 1 consists in a weir whose behaviour is close to a broad-crested weir, but whose hydraulic capacity is increased by reducing its width, rounding its usptream edges and endowing the spillways with a USBR profile.

The equation of the weir crest (1) is obtained from the USBR manual "Design of small dams":

$$y = 0.441x^{1.84} \quad (1)$$

The shape of the terminal structures (solid bucket for energy dissipation and length of the river bed to be protected) is obtained from the *Guía Técnica 5 de Seguridad de Presas* (Technical Guide nº5 of Dams Security), Spanish National Committee of Big Dams.

Their hydraulic study leads to conclusions that are summarized in the following table. For each case the discharge coefficient Cd was obtained as well as the discharge over the weir in two cases: with the full river section (strict hydraulic capacity, y = +2.5) and in the case of downstream submergence operation (water surface elevation equal to 3 m).

Therefore with alternatives 0 and 1 a significant increase of hydraulic capacity is achieved, hence the option of maintaining the current hydraulic profile is discarded.

Finally, the preferred option is to use a broad-crested weir (alternative 1), which simultaneously satisfies two important requirements:

- A noticeable increase of the hydraulic capacity in conditions of non-submergence.
- It eases the weir maintenance and operation in similar conditions to the current ones (this condition is not satisfied with the alternative 0).

It is important to note that for higher flow rates, the hydraulic efficiency gained with the new weir shape is progressively lost until it becomes equal to the current one for a discharge of 470 m^3/s (return period of 5 to 10 years), due to the influence of the downstream level that produces the dam submergence. In both cases the flow takes place simultaneously over the dam and the banks.

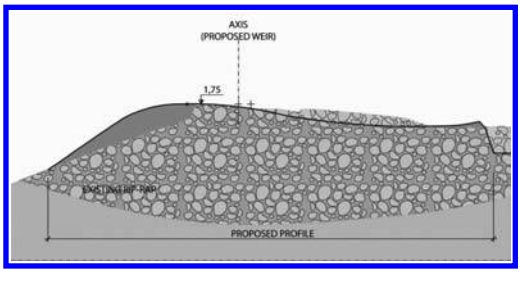

Figure 5. Alternative of maintaining a hydraulic behaviour identical to the current one.

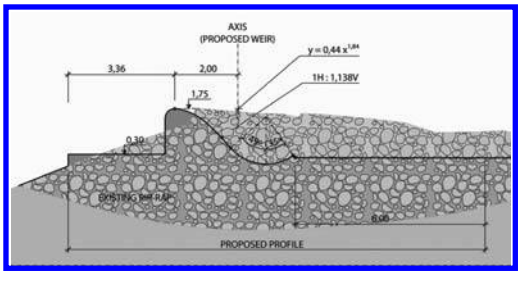

Figure 6. Alternative 0 for the weir shape.

Table 2. Results of the hydraulic analysis.

Flow (m³/s)	Present Weir (Cd:1.35)	Alt. 0 (Cd:2.18)	Alt. 1 (Cd:1.65)
Strict capacity	220	370 (+68%)	355 (+61%)
Extended capacity (DS submergence)	470	470	470

3.2 *Geotechnical analysis*

The geotechnical study carried out showed the following conclusions:

- Weir location: the decision of maintaining the current site is taken. The main reason is that thus we act on a soil at least partially consolidated and without causing a load increase with respect to the current situation, greatly simplifying the adopted foundation solution in comparison with a change of location.
- Analysis of leakages/ piping: in this calculation the length of the sheet piles is conditioned solely by the imperviousness of the strata under the weir. The following two cases have been studied:
  1. Situation before the execution of the sheet piles upstream and downstream of the dam: the leakage flow makes pumping compulsory. Also in this situation there is a risk of piping leading to execute a drainage ditch downstream of the dam so that by pumping the water table is lowered under the maximum digging height (–1.5 m). In this situation the estimated discharge was 300 l/s.
  2. Situation after the execution of the sheet piles upstream and downstream of the dam: in order to prevent the piping, it is enough to embed the piles in the grayish clayey silt (level –6 m), also obtaining a considerable reduction of the infiltration flow.

In addition to the conditions set out, there are additional ones to be taken into account when considering the alternatives for foundation and waterproofing:

- There is a large bed scour upstream of the dam reaching a minimum height of –6 m. For this reason and in order to ensure the transmission of stresses to a competent stratum it is necessary to embed the piles at a height of –12.
- Scour occurs during floods also downstream of the dam, therefore a second sheet pile must be proposed to control downstream erosion.

As a conclusion, the proposed foundation and watertight sealing method consists either on a flexible sheet pile made with bentonite-cement or on a structural sheet pile of reinforced concrete:

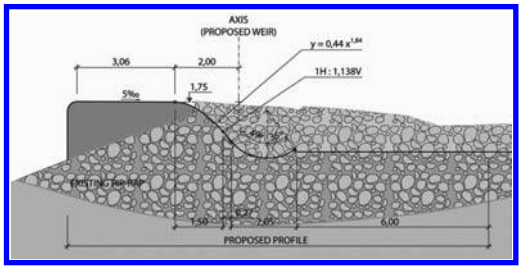

Figure 7. Alternative 1 for the weir shape.

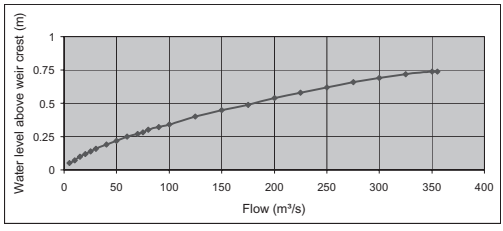

Figure 8. Flow and water level. Alternative 1.

- downstream of the weir down to –6 m, embedded in the impervious layer of grayish clayey silt.
- Upstream of the dam down to –12 m, embedded in the most competent layer of silty brown clays.

It is necessary to emphasize that from a geotechnical point of view the bentonite-cement sheet piles are more suitable, because they are not as stiff as the concrete ones and therefore they are best suited to adapt to any movements or future differential settlements, thus ensuring the imperviousness.

Finally, due to the scour and leakage detected in the study area, these sheet piles should be extended laterally beyond the dam layout in plan, in the direction of the axis of the "peninsula" (right bank) as well as towards the diversion channel (left bank).

3.3 *Structure analysis*

The following checks have been done:

- The dam design must allow the placement of common steel bars in the building practice considering the worst working conditions of the structure, taking also into consideration that at "long term" the dam can be barefoot in some areas and therefore it will work as a beam partially resting on the ground. The carried analysis leads to propose a minimum width for the weir of 1.2 m.
- The load transmitted by the dam and the sheet piles to the foundation soil is admissible.

Considering that on the same site there is already a dam resting on the river bed, we have checked that the load introduced by the proposed weir is smaller than the current one, which rules out the need of a structural foundation. Therefore the use of bentonite-cement sheet-piles is enough to ensure the required functions of water sealing and avoiding piping.

4 SELECTION OF THE BUILDING PROCEDURE

Based on technical and environmental criteria are selected the riverbank of the diversion channel (by using the existing channel in the left bank) and the kind of watertight enclosure, made with sheet pile cofferdam in order to minimize the river area affected by the works.

The performance in one or two stages is also analysed. Three alternatives are studied in this case, which are defined in terms of the maximum flow in the *Júcar* not affecting the watertight work enclosure and the number of stages proposed to execute the dam.

It is important to remark that the usual criteria given to obtain the diversion flood would compel to create a diversion channel with a bigger capacity than the river itself. Therefore, in order to obtain the diversion flood in this case, the river flow records have been analyzed from 1989 to our days, designing the diversion channel such as in average there will be an overflow over the watertight enclosure only 5–6% of the days during the works.

The following conclusions emerge from the analysis:

- Dam execution in a single phase, with a crest elevation of the sheet pile cofferdam (watertight enclosure) of 1.97 m.
- Proposal of a temporary diversion channel with a capacity of 30 m^3/s below the crest elevation of the sheet pile cofferdam, and regulated by gates to ensure that there is no water diversion below the level of operation of the existing upstream intakes.
- The provisional channel is afterwards used as a permanent diversion channel with a capacity (for a water elevation in the *Júcar* of +2.5 m) of 30 m^3/s, regulated by the same gates. The advantages are clear:
  1. it means slightly increasing the river hydraulic capacity in the Marquesa Weir (more than 10% of the current one);
  2. It also allows the increase of the flow through the fish pass during flood episodes, simulating the natural dynamics of rivers and therefore having a positive impact on its ecological functioning, such as natural cleaning, recovery of the lotic conditions after the growth of submerged macrophytes and other processes that contribute to habitat creation and maintenance.
  3. It facilitates future maintenance of the dam. The channel intake regulation and control through the gates employed during the work allows:
  4. to ensure the proper operation of the existing intakes upstream of the weir.
  5. To control the reservoir level upstream of the dam, thus assuming the role of a deep outlet, of an intermediate type.

5 PROJECT DESCRIPTION

5.1 *Phase I: Temporary diversion*

At this stage the following works are executed:

1. Earthfill cofferdams in intake and outlet channel cross-sections, anticipating the necessary pumping. The proposed cofferdam A.1 (see Figure 9) is common both for the diversion canal and the ecological canal, acting simultaneously as a fish pass. In order to build cofferdam (A.2) that facilitates the construction of the ancillary diversion under the island, it is previously necessary to build a temporary bridge over the diversion channel (D); it will facilitate both the access to the island and to the left abutment of the dam.

   The earthfill cofferdams will be made with material proceeding from the former digging, selecting the thinnest and most plastic fraction for the less pervious core, with a minimum thickness of 3 m in crown and slopes 1H: 2V. They will be completed with shells with a slope of 3H: 2V; material in both cases will be compacted until 95% of the MP.
2. Extension of the bentonite-cement upstream sheet pile on the left bank of the dam (B), exceeding the layout of the temporary diversion channel as well as the wall that envelops part of its length. Near the dam the crowning of the wall is given a slope of 11.5%, ranging in height from 1.75 (the dam crest elevation itself) to 2.5 m (the margin elevation), and then maintaining this value. This facilitates future access for maintenance equipment and personnel.
3. Provisional channel excavation (including intake canal C.1, transitions, gates section C.2 and C.3 diversion channel), channel coatings and gates installation.

   Its axis in plan coincides with the current existing channel in the *Júcar* left bank between PK 0 + 000 and 0 + 240. The proposed canal has a length of 399.54 m between the intake and the downstream connection to the river,

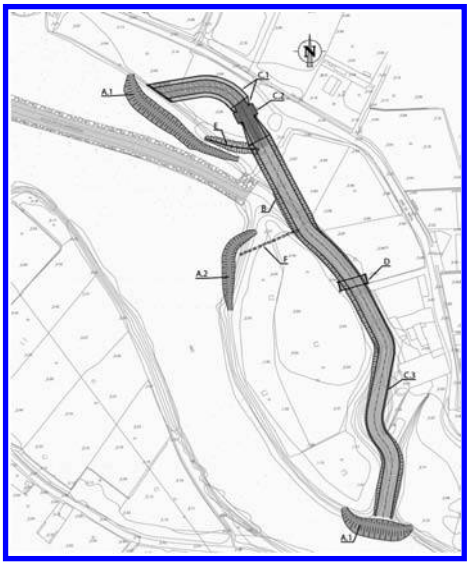

Figure 9. Temporary diversion works.

a slope of 0.2%, and a bed elevation of 0.45 upstream and of –0.344 in the final section.

4. The construction of the intake and the channel (E) with two scopes: to be used as the fish-pass intake and to allow the transit of the estimated ecological flow. It includes the fix lip weir, the connection with the diversion channel and the footbridge to easy the passage over the ecological canal. It is oriented so as to facilitate the entry of water, the length obtained is 40 m.

In both the intake and outlet of the diversion channel, the ecological channel and the auxiliary bypass, margins are protected with riprap.

This phase is completed with the removal of the three earthfill cofferdams employed, so that, after adjustment of the gates, the diversion channel, fish passage and the ancillary diversion are operating.

5.2 *Phase II: Dam execution*

At this stage the following works are executed:

1. The watertight enclosure required for the dam construction is created, first through the placement of the upstream steel piling (coffer dam), A in figure 10, using as working and driving platform both the existing dam and a temporary filling. After that the riprap protection downstream is placed, B, until it reaches an elevation of +1.00; next to it a temporary filling is executed, so that both make up a working platform from which to drive the steel piling-end of the watertight enclosure, C in figure 10. The enclosure is drained by pumping, allowing the execution of the drainage trench and the pumping ditch, D. The trench is dimensioned so as to prevent uplift in the working area in the most unfavourable conditions, i.e. before the implementation of the bentonite-cement sheet piles.
2. Dam execution, including: partial demolition of the current weir, E1, in order to execute the sheet piles first E2 (including the extension on the right bank in the case of the upstream one). After that the rest of the dam section E3 will be built, with a length of 216.07 m. The lateral closing walls E4 are included with a crest elevation of 1.75 m, as well as the wall that envelops the lateral extension of the sheet pile in the left bank and the access ramp to the dam for maintenance vehicles in the same margin, E5. It is projected with a slope of 11.5%, with an elevation ranging from 2.5 to 1.75.
3. In both abutments, plots are filled up to elevation +2.5 m, so that the flow is confined over the weir for the hydraulic conditions of strict river capacity, and rip-rap is put on the river margins and slopes exposed to the flow.
4. River bed protection F (right bank area) with rip-rap laying on a geotextile.

This second phase ends with the withdrawal of:

– The pumping equipment.
– The watertight enclosure (steel piling).
– The temporary fillings for driving/removing sheet piles and excess of rip-rap, which is destined to the abutments protection.

5.3 *Phase III: Permanent diversion*

The final deviation is obtained from the provisional one, reducing the widths in the necessary areas and adopting the final coatings. It requires the execution of partial flow diversions using part of the section of the channel itself.

This phase includes the demolition of the temporary structure and the construction of the pedestrian bridge to facilitate the access to the island.

5.4 *Restoration of the river environment*

Restoration of the river environment includes recovery of the riparian area, geomorphological adaptation of the banks, permeability of the dam to insure the fish passage, floristic diversification using bank native species and suitableness for public enjoyment.

5.4.1 *Elimination of invasive species*

We proceed to the elimination of invasive alien plant species as giant reed (*Arundo donax*), which

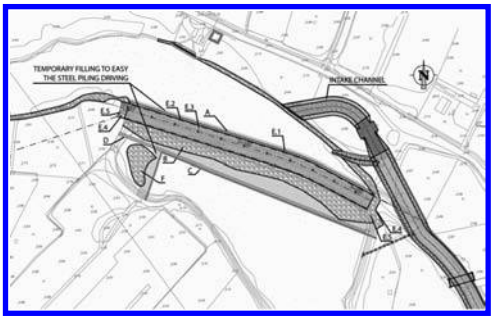

Figure 10. Phase II: Dam execution.

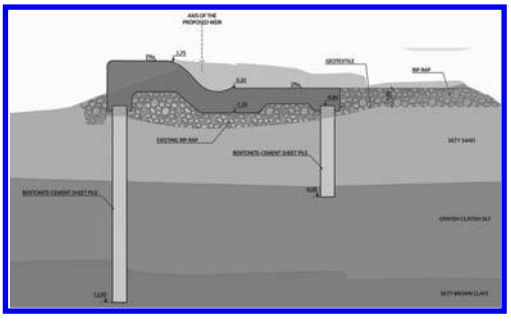

Figure 11. Weir cross-section.

presence is massive in the environment of the dam. Its elimination is particularly costly because of its vegetative reproduction by rhizomes that can reach down to 1.5 m depth.

In the months in which the translocation of nutrients from the leaves to the roots takes place, aerial part is triturated, rhizome is removed and all together is transported to landfill. The subsequent regrowth control is performed through the application of systemic insecticide (glyphosate) when rizhomes reach approximately 1 m height.

After removing the rhizome, the original shore morphology is restored with topsoil supplied from nearby fields.

5.4.2 *Longitudinal continuity*

The E channel (see Figure 9) and the diversion channel are conditioned to fulfil their ultimate function: allowing fish passage. In the final phase of the works (III) the diversion channel width is reduced at the base to 3 m, in the lowest 0.5 m to achieve adequate water depth with two thirds of the estimated minimum ecological flow, as shown in Figure 12. The slope of 0.2% is maintained. In order to create resting areas and to naturalize the channel, rip-rap stones of 50 cm diameter are placed dispersed on the channel bed with an average separation of 1.5 m. To stabilize the banks vegetated fiber rolls will be installed.

An example of such kind of channels is shown in next picture.

Since water does not exceed the weir crest (when flow is limited to the ecological flow), "pull effect" is guaranteed at the junction of the fish pass with the river, downstream of the weir.

5.4.3 *Plantations*

River banks and river shores are provided with adequate coverage and composition of native riparian vegetation.

River shores

Along river shores a macrophytes band is created with the following species: common reed (*Phragmites australis*), narrow leaf cattail (*Typha angustifolia*) and pale yellow iris (*Iris pseudacorus*). Next to the water surface and in order to encourage the river banks consolidation, these species are incorporated as part of fiber rolls.

First band of the river bank

It is a combination of riparian shrubs and trees where species adapted to wet soil near the river shores pre-dominate.

Shrubs: rosemary willow (*Salix eleagnos*), black willow (*Salix atrocinerea*), purple willow (*Salix purpurea*), oleander (*Nerium oleander*), and tamarisk (*Tamarix gallica*).

Trees: white willow (*Salix alba*) and white poplar (*Populus alba*).

Second band of the river bank

It is a combination of riparian shrubs and trees adapted to slightly less wet soils that the ones of the first band.

Shrubs: oleander (*Nerium oleander*), tamarisk (*Tamarix gallica*), hawthorn (*Crataegus monogyna*), "labiernago" (*Phillyrea angustifolia*) and Laurustinus (*Viburnum tinus*), the last two in the highest and most distant areas from the river bed.

Trees: white poplar (*Populus alba*) and hackberry (*Celtis australis*).

In the inner part of the peninsula located between the river and the channel, characteristic species of the most representative habitats of the Site of Community Importance (SCI) "Middle and Lower Course of *Júcar* River," are sown and planted, including: *Salix alba* and *Populus alba* galleries, Southern riparian galleries and thickets (*Nerio-Tamaricetea* and *Securinegiontinctoriae*), Mediterranean tall humid grasslands of the *Molinion-Holoschoenion* and Constantly flowing Mediterranean rivers with *Paspalo-Agrostidion*.

5.4.4 *Paths and equipment*

The social enjoyment of the environment should be used to spread the ecological, cultural and social values of the dam and the *Júcar* river banks to users and other visitors.

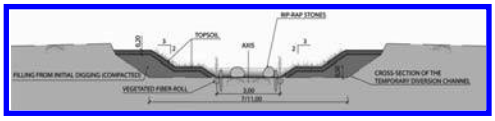

Figure 12. Lateral fish-pass channel, typical cross-section after phase III.

Figure 13. Example of a fish side channel.

Table 3. Works budget.

Chapter	Euros (€)
Temporary diversion channel, fish-pass	462,708.59
Dam reconstruction and banks protection	4,669,038.17
Final diversion channel	32,038.19
River banks restoration	671,160.63
Temporary structure and footbridge	130,215.79
Remote control system	112,891.65
Utilities	175,952.10
Environmental monitoring plan	40,976.65
Waste management	589,991.09
Health and security system	297,461.74
Total with overheads, benefit and taxes	10,247,897.69

Exclusively pedestrian paths are to be built along the river banks, in the areas of access to the piers and in the inner part of the peninsula in order to link the stands of the different SCI habitats. To give continuity to the network of tracks, paths start and meet the already existing service roads and the ones to be performed.

Tables with benches, bicycle parking and piers are to be installed, all made of wood, to achieve a greater integration in the natural ecosystem. Information panels are also to be installed with the relevant description of the main characteristics of the *Júcar* riverine ecosystems.

Finally, wooden crossbeams and steel chains are installed, in order to restrict unauthorized vehicle access into the *Júcar* river and banks.

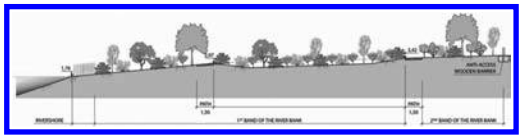

Figure 14. Plantations typical cross-section.

6 BUDGET AND EXECUTION TERM

The period of execution of the works is 24 months. The budget is summarized in the following table.

7 CONCLUSIONS

The successive river floods have damaged the *Marquesa* weir, which has been repaired by the addition of rip-rap and the replacement of the concrete shell. To ensure compliance with the dam current functions, which are preventing upstream saline intrusion in the *Júcar* river and insuring the performance of the existing intakes, its reconstruction is necessary.

The design has been performed by a multidisciplinary team, taking advantage of the opportunity to rebuild the dam to carry out broader action to restore the banks nearby, to create fish transit through a lateral fish passage and to increase the dam hydraulic capacity. Thus the proposed works become one of the first cases of application in the *Júcar* basin of the National Strategy for Restoring Rivers, as for the weir acquired permeability to the fish fauna passage and the restoration of the surrounding shores, complying therefore with the Water Framework Directive.

Reconstruction of the dam guarantees its current functions in the coming years and allows at the same time to improve its hydraulic performance, endowing it with greater hydraulic capacity and enabling its maintenance through the use of the side channel.

Both the solution and the construction procedure have been selected after technical and environmental analysis of the studied alternatives, and at all times ensure compliance with the dam current roles. As an example:

- the proposed construction procedure or the selected site for the reconstruction both minimize the affected river surface.
- Most of the temporary works are used with different scopes once the dam finished.
- The current hydraulic capacity is maintained during the works and improved after them, the separation between fresh and salt water is guaranteed as well as a proper functioning of the existing intakes. Finally the fish transit is restored from the beginning of the works.

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Proyecto de renovación de la toma del Canal del Cinca de la presa de Valdabra

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RESUMEN: El artículo describe las obras realizadas en la presa de Valdabra, situada en el término municipal de Huesca, para mejorar los cierres de la toma del Canal del Cinca mediante la colocación de doble cierre en cada uno de los cuatro conductos existentes, ya que previamente se disponía de un único cierre por conducto. De esta manera, se consigue mejorar la regulación de la Toma del Canal, dotándola de mayor seguridad al existir tres compuertas por conducto. La obra se completa con la construcción de una nueva caseta de mayores dimensiones para albergar las doce compuertas, los nuevos grupos oleohidráulicos de accionamiento y cuadros de mando y control–uno por cada dos conductos–, la instalación eléctrica de los accionamientos y un puente grúa.

ABSTRACT: The article describes the works carried out in Valdabra dam, located in the town of Huesca, to improve the closure of the Cinca Channel intake by placing double gate in each of the four existing channels, since they were previously equipped with a single closure. This way, better regulation of the Cinca Channel intake is achieved, getting better security since there are three gates by channel. The work was completed with the construction of a new larger valve house to accommodate the twelve gates, new oleo-hydraulic groups drive and control panels—one each two lines–, the electrical installation of drives and a overhead traveling crane.

1 INTRODUCCION

La Presa de Valdabra se encuentra en el barranco del mismo nombre, afluente del río Flumen. Tiene como finalidad regular el caudal de agua que circula por el Canal del Flumen, con destino al Canal de Monegros, con un recorrido aproximado de 89 km. Esta obra recoge la colocación de un doble cierre en cada conducto de la Toma del Canal del Cinca, para poder llevar a cabo una óptima y segura regulación de caudales.

Figura 1. Plano de situación de la presa de Valdabra.

2 DESCRIPCIÓN DE LAS PRESAS

La presa de Valdabra se halla al sur del término municipal de Huesca, y fue puesta en servicio en el año 1985. Es una presa de material homogéneo limo-arcilloso, con manto de escollera en el talud de aguas arriba y capa vegetal en el de aguas abajo. Tiene 20 m de altura sobre cimientos, 10 m sobre el cauce y 500 m de longitud de coronación.

Figura 2. Vista general de la presa de Valdabra.

La toma del Canal del Cinca se efectuaba con cuatro compuertas deslizantes de 2,75 × 2,50 m de accionamiento oleohidráulico. Estas compuertas, a pesar de que su puesta en servicio data de 1985, no disponen de un doble cierre en cada uno de los conductos. Esto imposibilitaba la realización de múltiples operaciones de mantenimiento preventivo, como puede ser una apertura total de compuerta.

A todo esto se le sumaba que la maniobra de apertura y cierre de las compuertas era muy lenta dadas sus dimensiones. La funcionalidad de las compuertas era incorrecta debido a aspectos negativos como los sedimentos de cal, diseño incorrecto de las propias compuertas (más anchas que altas, lo que provoca acodalamientos) y una falta de maniobras completas periódicas. Para corregir esta situación se pensó en la instalación de un doble cierre en cada uno de los conductos, con el fin de aportar fiabilidad y seguridad a la toma del Canal del Cinca.

Se colocaron 4 nuevas compuertas de seguridad y 4 nuevas compuertas de regulación a continuación de las actuales compuertas. Al mismo tiempo se realizaron mejoras en los accesos a las instalaciones (caminos asfaltados, iluminación, ...), en las propias instalaciones (construcción de nuevo edificio de compuertas), el abastecimiento de agua por medio de una bomba con características adecuadas, iluminación de la salida de agua de las compuertas de la toma y por último la adecuación del entorno de las obras.

2.1 *Presa de Valdabra*

La presa de Valdabra, situada al sur del término municipal de Huesca, tiene como finalidad regular el caudal de agua que circula por el canal del Flumen desde la presa de El Grado, con destino al Canal de Monegros en el llamado abrazo de Tardienta, con un recorrido aproximado de 89 km. Fue puesta en servicio en el año 1985.

Es una presa de materiales sueltos, que crea un embalse con una capacidad de 1,34 Hm^3 con NMN a la cota de 399,10 msnm.

Dispone de aliviadero de labio fijo en laberinto, de 258,00 m de longitud desarrollada, capaz de evacuar 135 m^3/s.

El embalse es alimentado por el propio Canal del Cinca. Justo en su embocadura se dispone de una almenara que está formada por una compuerta Vagón Transversal de 6,40 × 4,50 m con contrapeso y accionamiento mediante un motorreductor al cual se le ha acoplado un mecanismo desmultiplicador de velocidad. A su vez dispone aguas arriba de la compuerta transversal, de un desagüe del propio Canal, regulado por dos compuertas deslizantes de 0,80 × 0,80 m de accionamiento manual que desaguan en el cuenco amortiguador del aliviadero.

Figura 3. Vista del aliviadero de la presa de Valdabra.

La toma del Canal del Cinca se efectúa con cuatro compuertas deslizantes de 2,75 × 2,50 m de accionamiento oleohidráulico con su eje a la cota 392,82 msnm. Su capacidad de desagüe está limitada por el propio Canal a 43,45 m^3/s.

En el arranque del canal existen tres compuertas deslizantes de 0,60 × 0,60 m con una capacidad de desagüe de 5,91 m^3/s, que actúan como desagüe de fondo del embalse.

La desembocadura del Canal del Cinca y el aliviadero se encuentran en el estribo izquierdo y la toma y desagües de fondo en el derecho.

Al igual que en el Canal de alimentación, aguas abajo de la toma del Canal del Cinca existe otra almenara de idénticas características a la de entrada.

3 OBJETO DE LAS ACTUACIONES

A continuación se detallan las obras ejecutadas.

3.1 *Descripción de la toma del Canal del Cinca*

En planta, el agua procedente del Canal del Cinca ha de atravesar primero unas rejas de limpieza colocadas en cuatro vanos de 2,75 m. Cada reja está compuesta por dos módulos de 2,75 × 4,50 m, montadas de dos en dos en cada vano. Las rejas son de acero al carbono, tratadas con un tratamiento anticorrosivo.

A continuación existe un pequeño tramo de canal de aproximadamente 15,00 m de longitud con una sección de paso de 14,00 × 9,00 m.

Al final de este pequeño canal se encuentra un paramento en cuya zona inferior se encuentran

las cuatro únicas compuertas de regulación del Canal del Cinca. Este paramento es lo suficientemente estanco como para albergar aguas abajo la cámara de maniobras donde se ubican las válvulas de regulación.

Los órganos encargados de la regulación del Canal del Cinca eran cuatro compuertas deslizantes de 2,75 × 2,50 m posicionadas en paralelo, con lo cual sólo existía un cierre único para dicho Canal. Las cuatro compuertas deslizantes eran de acero al carbono y accionamiento oleohidráulico, no disponiendo de enclavamiento mecánico en la posición de apertura total de compuerta, aunque existía un enclavamiento oleohidráulico del tipo doble antirretorno pilotado.

Ninguna de estas cuatro compuertas disponía de by-pass, dado que se trataba de compuertas de regulación. Tampoco disponían de sistemas de aireación.

Las compuertas eran maniobradas mediante un grupo oleohidráulico, ubicado en la misma sala, que disponía de un distribuidor manual encargado del accionamiento de cada una de las compuertas.

Existía un cuadro de mando que activaba los dos motores de la central oleohidráulica. Disponía de indicador luminoso de motor en marcha, amperímetro, voltímetro y protecciones eléctricas mediante interruptores diferenciales y magnetotérmicos.

Aguas abajo de las compuertas de regulación del Canal del Cinca, existe una embocadura al canal de aproximadamente 24,00 m de longitud en la que sufre una notable disminución de anchura.

3.2 *Estado de la toma del Canal del Cinca*

Los conductos de las tomas del Canal del Cinca a pesar de no ser muy antiguos (su puesta en servicio data de 1985) no disponen de un doble cierre en cada uno de los conductos. Esto imposibilita la realización de múltiples operaciones de mantenimiento preventivo, como puede ser una apertura total de compuerta. A todo esto se le suma que la maniobra actual de apertura y cierre de las compuertas es muy lenta dadas las dimensiones de las mismas.

Estos problemas se pensaban que iban a desaparecer con la instalación de un nuevo grupo oleohidráulico (1994), que mejoraba el movimiento de las compuertas que hasta entonces se desplazaban muy lentamente y a tirones. Actualmente y debido a aspectos negativos tales como los sedimentos de cal, las dimensiones defectuosas de las propias compuertas (más anchas que altas, lo que provoca acodalamiento) y una falta de maniobras completas periódicas (tal y como recomiendan los programas de mantenimiento preventivo), la funcionalidad de las compuertas siguen siendo incorrecta.

Todos estos impedimentos hacen que sea prácticamente imposible una maniobra total de cada una de las cuatro compuertas, para lo cual se entiende necesaria la instalación de un doble cierre en cada uno de los conductos, con el fin de aportar fiabilidad y seguridad a la toma del Canal del Cinca.

El grupo oleohidráulico y cuadro de mando, aunque se les ha realizado mejoras, necesitan la sustitución de ciertos elementos ya que se deterioran rápidamente debido a las duras condiciones ambientales.

3.3 *Actuaciones ejecutadas*

Capítulo 1: Actuaciones en el embalse.
Capítulo 2: Reparaciones.
Capítulo 3: Demoliciones y adecuación de la caseta.
Capítulo 4: Nueva instalación.
Capítulo 5: Instalaciones eléctricas e iluminación.

Figura 4. Vista de la caseta de la toma del Canal del Cinca antes de las obras.

Figura 5. Vista de la caseta de la toma del Canal del Cinca después de las obras.

Capítulo 6: Adecuación de los accesos.
Capítulo 7: Obras varias.

4 DESCRIPCIÓN DE LOS TRABAJOS A REALIZAR

Seguidamente, se describen las obras proyectadas.

4.1 *Actuaciones en el embalse*

Previa a cualquier actuación en el embalse, es necesaria una toma de datos general de la situación y una inspección detallada de cada una de las instalaciones a modificar, corregir o sustituir.

Para ello se procedió, por parte de personal técnico especializado, al a inspección y toma de datos de los cajeros de las rejas y ataguías para posteriormente diseñar y fabricar las 4 ataguías correspondientes para dejar en seco en cuenco de la Toma del Canal.

4.2 *Reparaciones*

Tras la actuación anterior, se desmontaron las cuatro rejas para transporte a taller para su limpieza, reparación y tratamiento anticorrosivo. Una vez finalizada la obra y antes de la retirada de las ataguía se procedió al montaje las rejas.

Las compuertas deslizantes, que a partir de ahora funcionaran como ataguías de seguridad, y sus instalaciones eléctricas y oleohidráulicas necesitaron de una revisión, sustituyendo los elementos que se encontraban deteriorados.

4.3 *Demoliciones y adecuación de la caseta*

Se amplió la caseta para poder albergar las nuevas compuertas, teniendo unas dimensiones finales de $14{,}80 \times 7{,}60 \times 8{,}94$ m, realizando las actuaciones necesarias para ello, incluyendo la construcción de pilares, techo y ménsulas para soportar un nuevo puente grúa.

Dado que las nuevas compuertas son de menores dimensiones, $1{,}10 \times 1{,}80$ m, se acondicionaron los tajamares para su unión con el hormigón de recubrimiento.

La construcción de la caseta se completó con un forjado formado por losas alveolares, una capa de mortero para formación de pendientes, una capa de EPDM como impermeabilización y un solado con baldosas térmicas. Finalmente, se terminaron elementos interiores como escaleras para acceder al nivel de las nuevas compuertas, ventanas, etc.

Se acondicionaron las laderas del canal para evitar derrumbamientos y la solera y los laterales del mismo.

4.4 *Nueva instalación*

La modificación de la Toma del Canal del Cinca se realizó prolongando cada uno de los conductos. Se procedió a la preparación de los hierros fijos existente para la colocación de la transición a las nuevas compuertas. La prolongación se ejecutó mediante una transición de las actuales compuertas, modificando la sección de paso de $2{,}75 \times 2{,}50$ m a $1{,}10 \times 1{,}80$ m. A continuación de esta transición se incorporaron dos compuertas tipo "Bureau".

En los conductos de los extremos izquierdo y derecho, dada la geometría del canal, la transición es oblicua de unos 11º aproximadamente con el fin de que el volumen de agua saliente guarde linealidad con los laterales del cauce del canal.

Las compuertas de seguridad disponen de un sistema de enclavamiento en su posición de máxima apertura, ya que este tipo de compuertas se encuentran prácticamente en posición de apertura máxima cuando entran en explotación, al mismo tiempo dispondrán de un by-pass con doble cierre y un sistema de aireación (ventosa multifuncional) que permita tanto el suministro de aire para favorecer la dinámica del fluido como la expulsión al exterior del aire que pueda acumularse en el interior del conducto.

Las válvulas de regulación tendrán de un sistema de aireación capaz de suministrar aire en la salida del conducto favoreciendo la dinámica del fluido.

Para el accionamiento de las compuertas se dispone de dos grupos motobombas y dos cuadro de mando y protección, uno para cada dos conductos, con todas las instalaciones oleohidráulicas y eléctricas de conexión necesarias.

Las instalaciones se completaron con un sistema de by-passes con doble cierre en las válvulas de seguridad, un carrete de salida tras el cierre de regulación, un sistema de aireación mediante ventosa multifuncionales con su válvula de seguridad, para cada compuerta de seguridad y una tubería de aireación para cada compuerta de regulación.

4.5 *Instalaciones eléctricas e iluminación*

Se incorporó una nueva línea de fuerza especialmente dimensionada y diseñada para el mando de las antiguas compuertas, las ocho nuevas válvulas y el sistema de puente grúa. El suministro de potencia cubre el sistema de iluminación del interior de la propia cámara, ya que se verá incrementada en dimensiones.

La instalación eléctrica se completó con la instalación del cableado necesario de conexión a la línea de fuerza y de iluminación, un cuadro de

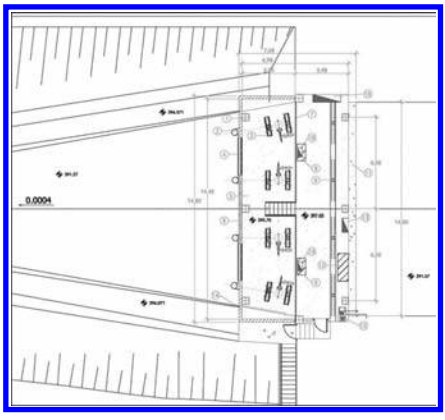

Figura 6. Planta de la toma del Canal del Cinca.

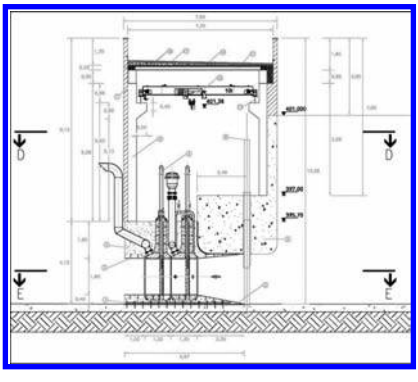

Figura 7. Perfil de la toma del Canal del Cinca.

distribución de la línea de fuerza dentro de la cámara de válvulas, una caja anexa al cuadro general de la cámara con los elementos necesarios como enchufes y protecciones de seguridad, luminarias interiores y exteriores con las conexiones necesarias.

4.6 *Adecuación de los accesos*

Se ejecutaron mejoras en los accesos a las instalaciones, mejorando el acceso a la cámara de maniobras por ambas márgenes. También se repuso todo el vallado necesario y las barandillas de seguridad.

4.7 *Obras varias*

Para completar las obras se procedió a la sustitución de la bomba existente por otra de iguales características, y se reubicó dentro de la nueva caseta para protegerla del tiempo.

El conjunto de las actuaciones de la obra no afectó a la campaña de riego, terminándose el resto de actuaciones en la campaña de riego.

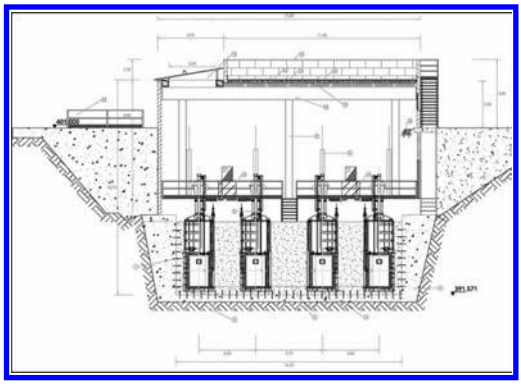

Figura 8. Alzado de la toma del Canal del Cinca.

5 CONCLUSIONES

Con la ejecución de las obras de renovación de la Toma del Canal de Cinca, se logra una mejora en la funcionalidad y seguridad en la presa ya que dispone de doble cierre por conducto, adaptándose al reglamento vigente proporcionando seguridad en la explotación y en la gestión de laminación de avenidas.

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Sustitución de los equipos electromecánicos e hidromecánicos de los desagües de las presas de Amadorio, Beniarrés y Guadalest (Alicante) y Forata (Valencia)

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ABSTRACT: The purpose of the projects is the replacement of electromechanical and hydraulic elements of the Bottom Outlets, Intake Structures and Spillway of several dams maneged by the Jucar River Basin Authority.
BACKGROUND: The construction of the dams Amadorio, Beniarres, Guadalest and Forata was conducted between the years 1950–1970. Since then, several maintenance works have been made to make sure these facilities were in good operational state.
SCOPE OF THE WORKS: The work includes the replacement of the elements that are in poor condition and repair of those which can be recoverable. On the other hand has been found by bathymetry conducted in reservoirs, that is not possible to replacement of the upstream gates of the bottom outlet since it is required before removal of the existing sediment that obstruct the mouths. The works had to be the divided into two phases or project the because of the need for dredging the sediments. Consequently, the replacement of the downstream gates of the bottom outlet were envisioned in the first project, and the second project included the removal of sediments and the replacement of the upstream gates. This second phase and project has to be submitted to the consideration and approval to the environmental Authorities in order to guarantee the compatibility of the dredging of the sediments with the environment. At the present time different alternatives are being analyzed to provide a viable solution considering environmental constraints. In parallel, the first phase of the projects are being implemented at the present.
SPILLWAY: It is carrying out the replacement of Galle chains, guide rollers, parts of drive mechanism and the waterproofing of the Taintor gates and the provision of a new redundant system drive.
BOTTOM OUTLET: The downstream Bureau gates of the two pipes are being renewed, although the embedded bodies in concrete are kept, while boards are replaced. The central hydraulic power and control panel are also replaced.
INTAKE WATER: In this case as it is not necessary dredging of sediments from the mouth, are being replacing both the floodgates upstream and downstream. This requires underwater work for the placement of provisional gates to seal the conducts to work in dry conditions.

1 ANTECEDENTES

Dada la antigüedad de los equipos de las presas indicadas, y aunque ha existido un programa de mantenimiento, se ha manifestado necesario una profunda revisión de los mismos para garantizar su perfecto funcionamiento. Los trabajos contemplan, por tanto, la sustitución de los elementos que se encuentran en mal estado, y la revisión y reparación de aquellos elementos que sean recuperables.

Inicialmente las actuaciones de sustitución de los equipos electromecánicos e hidromecánicos de los desagües de las presas de Amadorio, Beniarrés, Guadalest y Forata no incluían el estudio de lodos del embalse, cuyo dragado, junto a los desagües de fondo, era imprescindible acometer para la reparación de las compuertas de aguas arriba.

Consecuentemente con lo anterior, los desglosados nº 1 de los trabajos de adecuación de los equipos electromecánicos e hidromecánicos de los desagües de las presas de Amadorio, Beniarrés, Guadalest y Forata, incluyen todas las actuaciones sobre elementos hidromecánicos de la presa que se puedan realizar sin tener que efectuarse dragado de lodos.

2 RELACIÓN DE TRABAJOS REALIZADOS

Las obras y trabajos objeto de las actuaciones se centran en los aliviaderos, desagües de fondo y tomas de agua, según se describe seguidamente.

También se contempla, en la presa de Beniarrés, la adecuación de la plataforma y carriles del montacargas de plano inclinado de la toma.

2.1 *Aliviaderos*

La reparación de los aliviaderos ha consistido básicamente en la sustitución de los mecanismos de elevación, de las cadenas Galle, de los rodillos guías laterales, de las impermeabilizaciones y de los hierros fijos de solera. Asimismo, se han montado deslizaderas de acero inoxidable en los hierros fijos laterales y regletas indicadoras de la posición de las compuertas. Se han instalado también grupos autónomos con motor diesel y bomba hidráulica, para el accionamiento en caso de emergencia de las compuertas Taintor por medio de un motor hidráulico acoplado a la columna de accionamiento alternativo del mecanismo. Por último, se ha procedido a realizar una protección con pintura de todos los elementos metálicos que no han sido sustituidos.

Vistas del estado inicial de compuertas del aliviadero de la Presa de Guadalest.

2.2 *Desagües de fondo*

Las reparaciones en los desagües de fondo han consistido en la sustitución de los puentes grúa por otros de accionamiento eléctrico, desmontaje de las compuertas Bureau de aguas abajo, sustitución de los tableros y reparación de las tapas y de los cierres del cuerpo, nuevos cilindros con sus correspondientes grupos hidráulicos, etc. Los grupos

Estado inicial compuertas Bureau (Beniarrés).

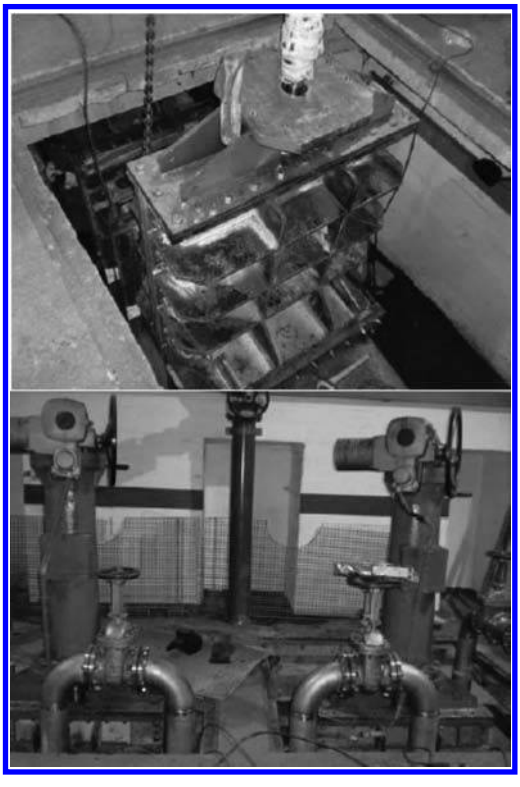
oleohidráulicos existentes se han mantenido para el accionamiento de las compuertas de seguridad, aunque se han colocado pupitres electrohidráulicos nuevos para el accionamiento de los mandos, controles y señalización de cada una de las compuertas Bureau. Para las compuertas de seguridad se han realizado nuevas aducciones conectadas a las de aireación de las Bureau de regulación. Por último, se ha colocado el piso de entramado metálico nuevo y se ha realizado una protección con pintura de todos los elementos metálicos que no han sido sustituidos.

2.3 *Tomas de agua*

La reparación de las tomas de agua consiste básicamente en la sustitución de los mecanismos de las compuertas de paramento y de las rejas, así como la reparación de las barras y rodillos de transmisión. En la cámara interior se han respetado las válvulas de compuerta, sustituyendo los cilindros de accionamiento. El resto de equipos se ha sustituido montando en el lugar de las válvulas, compuertas Bureau. Los puentes grúa se han sustituido igualmente por otros de accionamiento eléctrico. A continuación se enumeran cada una de las operaciones o suministros que definen la reparación:

2.3.1 *Trabajos Subacuáticos*

La realización de los trabajos de sustitución de todos los elementos, y en su caso los de las compuertas de seguridad, se ha realizado con el embalse lleno, por lo que se ha necesitado la ayuda de un equipo de buceo con submarinistas especializados.



Montaje de compuerta de seguridad en la toma de agua (Forata).

Los trabajos subacuáticos a realizar en las tomas de agua se resumen en un tapón a colocar en la toma de agua del equipo de desentarquinar, montaje y fijación de escudos, retacado de las juntas, desmontaje de la placa y colocación de reja que cubra la embocadura.

Para cada una de las fases enumeradas, se ha previsto, a nivel presupuestario, la oportuna movilización y desmovilización de personal, equipo y materiales.

2.3.2 *Compuertas de paramento*

Suministro y montaje de los mecanismos para las compuertas de paramento constituido por un motorreductor cerrado protección IP67, husillo ascendente, indicador de posición, limitadores de par en ambos sentidos, finales de carrera, columna de accionamiento y todos los equipos y accesorios necesarios para el correcto funcionamiento, incluido el desmontaje de los existentes y el transporte a vertedero.

Revisión y reparación de las barras de transmisión y los rodillos guía de elevación de las compuertas de paramento.

Instantáneas de una de las fases de trabajos subacuáticos, realizados mediante el aparataje de cordón umbilical y con transmisión en directo, visual y acústica, de la operación (Beniarrés).

Revisión y reparación de los elementos metálicos y posterior pintura de protección de la totalidad de las rejas actuales.

Se ha aplicado una pintura de protección a todos los elementos metálicos que no han sido sustituidos.

Estado de las rejas tras el proceso de renovación realizado (Guadalest).

2.3.3 *Compuertas cámara interior presas*

Los principales trabajos realizados han sido los siguientes:

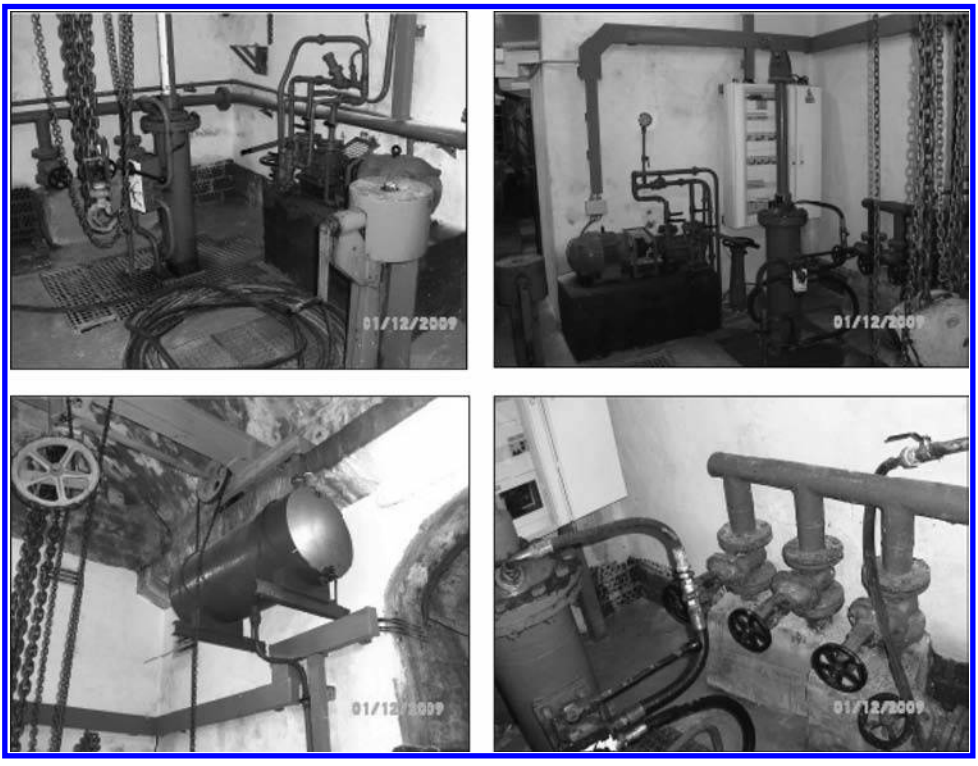
Desmontaje de los puentes grúa manual y sus carriles existentes y suministro y montaje de carriles de rodadura, así como de puentes grúa para una capacidad de elevación de 2.5 T.

Hormigonado de las tuberías del sistema de desentarquinar en las fases de reparación de cada uno de los conductos, para su completa anulación.

Reparación de los cierres de las válvulas de compuertas. Sustitución de cilindros hidráulicos, by-Pass para cada conducto, aducciones de aire

para las válvulas de compuerta, carretes intermedios, carretes de transición, compuertas deslizantes, pupitres electrohidráulicos, aducciones de aire con válvulas de mariposa y ventosas bifuncionales, y pintura de protección para todos los elementos metálicos que no han sido sustituidos. Todos los elementos, tubos, codos, bridas, válvulas y tornillería son de acero inoxidable.

El estado inicial de las cámaras de tomas era el que se observa seguidamente:



Cámara de toma de Guadalest.

3 DESCRIPCIÓN DE LAS OBRAS MÁS SIGNIFICATIVAS

3.1 *Presa de forata*

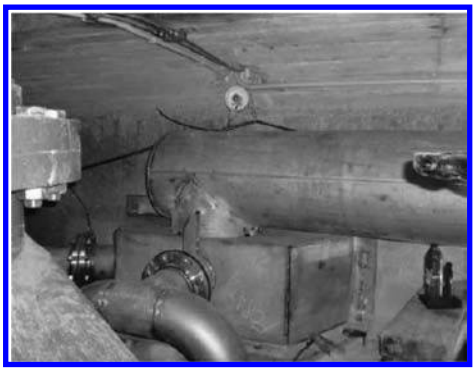
3.1.1 *Sustitución de compuertas de seguridad de la toma de agua*

En primer lugar se han extraído las rejas existentes de su ubicación en la entrada de los conductos, utilizando la viga tenaza instalada en la caseta de paramento de la presa y una grúa auxiliar.

A continuación, el equipo de buceo limpió los escombros de las embocaduras y tomó medidas de los huecos de los conductos para la fabricación del escudo que permitió trabajar en seco en la cámara de compuertas.

Una vez fabricado e instalado el escudo se desmontó la válvula de seguridad y sus elementos de by-pass y aducción, sustituyéndose la compuerta Bureau.

3.1.2 *Caja de vientos en desagüe de fondo y túnel de desvío*



Caja de vientos instalada en cámara de compuertas.

En las cámaras se ha optado por la opción de instalar una caja de acero inoxidable en el exterior del hormigón, apoyada en la superficie del suelo de la cámara de compuertas.

Para apoyo de la caja sobre el hormigón e impermealización de esta zona se ha instalado una junta de goma.

Esta caja se comunica con cada uno de los conductos de salida de agua por medio de perforaciones realizadas en el suelo de la cámara, justo sobre la vertical de los conductos.

Perforaciones sobre la clave de las galerías de fondo para conectar a la caja de vientos las aducciones aguas abajo de las compuertas Bureau.

Se realizaron perforaciones en el desagüe de fondo y el túnel de desvío.

En estas cajas se conectan las tuberías de la aireación antigua de las compuertas de regulación, por lo que se aumenta la superficie de los tubos en comunicación con cada conducto. Igualmente, se han comunicado con la caja las tuberías de las compuertas de seguridad.

Esta solución ha sido adoptada en el resto de embalses por lo que no se volverá a comentar en los apartados correspondientes.

3.1.3 *Limpieza del conducto de salida del desagüe de fondo*

Los conductos de los desagües de fondo están obstruidos por el barro y fango acumulado. Lo mismo sucedía con las tuberías del by-pass de las compuertas.

Al desmontar el tablero se limpia, con medios mecánicos, el barro del cuerpo de la compuerta Bureau de regulación y se coloca una manguera con agua para intentar diluirlo, de manera que este fluya por el conducto, operación que se llevó a cabo sin éxito.

Medios empleados para la limpieza de fangos.

A continuación se optó por inyectar agua a presión en el conducto, resultando de manera insatisfactoria.

Por último, se accedió a la zona exterior del conducto aguas abajo, utilizando una barca para pasar a través del cuenco amortiguador. Para el desarrollo de los trabajos se ha instaló un sistema de ventilación forzada y se emplearon detectores de gases de forma permanente.

3.1.4 *Túnel de desvío*

Los trabajos realizados en el túnel de desvío de Forata han sido los siguientes:

Sustitución de compuerta Bureau de regulación, de dimensiones 1,25 × 1,50 m. Se sustituye el tablero y el servomotor de accionamiento, y se repara la tapa de la compuerta.

3.2 *Presa de amadorio*

3.2.1 *Sustitución de compuertas de las toma de agua*

En primer lugar se han extraído las rejas existentes de su ubicación en la entrada de los conductos, utilizando la viga tenaza instalada en las casetas de paramento de la presa para realizar a continuación las intervenciones previstas tanto en la compuerta de paramento como, a continuación, cerrada dicha compuerta, en la sustitución de todos los elementos de las tomas.

Valvulería y Calderería acero inoxidable.

3.2.2 *Desagüe de fondo*

El desagüe de fondo no resulta totalmente estanco puesto que la compuerta de seguridad presenta fugas en la parte inferior del tablero. Por este motivo no se ha podido realizar la reposición de los cierres laterales y el dintel del cuerpo de las compuertas bureau, y sustitución del metal babitt del cierre del umbral donde se deberá actuar cuando se realice la adecuación de las compuertas de seguridad.

3.3 *Presa de guadalest*

3.3.1 *Trabajos subacuáticos*

Los trabajos subacuáticos a realizar en las tomas de agua de Guadalest se resumen en un tapón a colocar en la toma de agua del equipo desentarquinador.

3.3.2 *Limpieza del conducto de salida del desagüe de fondo*

Los conductos de los desagües de fondo están obstruidos por el barro y fango acumulado. Lo mismo sucede con las tuberías del by-pass de las compuertas. Se ha previsto la limpieza del conducto mediante la utilización de una lanza de agua de alta presión accediendo desde aguas abajo por los conductos de desagüe.

Desagüe de fondo Amadorio.

El acceso a la zona exterior del conducto aguas abajo, se realiza utilizando una barca a través del cuenco amortiguador y por medio de una pasarela de acceso adosada al paramento de la presa.

En cuanto a la compuerta de seguridad del desagüe de fondo del conducto derecho se encuentra atascada. Por dicho motivo se ha previsto sustituir el cilindro hidráulico existente y tratar de accionarla, cerrándola completamente, para poder actuar posteriormente sobre la de aguas abajo y realizar la sustitución de su tablero, cierres, sistema de aducción y by-pass previstos.

Bureau toma Guadalest.

3.4 *Presa de beniarres*

3.4.1 *Trabajos subacuáticos*

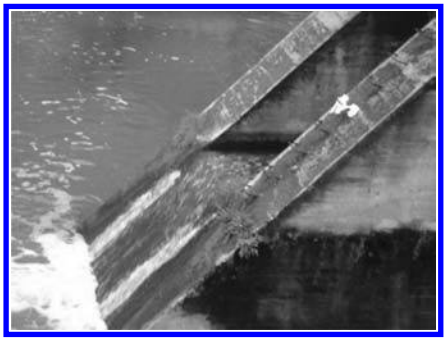
En Beniarrés, los trabajos subacuáticos se pueden resumir en tres fases: Montaje y fijación de la placa o escudo por medio de taco o expansión, en una embocadura y retacado de las juntas del escudo; Desmontaje del escudo, montaje en la otra embocadura, y retacado de las juntas del escudo; Desmontaje del escudo, colocación de una reja que cubra las dos embocaduras e izado a la superficie del escudo.

3.4.2 *Desagüe de fondo*

El desagüe de fondo no resulta totalmente estanco presentando abundantes fugas tanto por el tablero como por el by-pass, la compuerta de seguridad presenta fugas en los cierres. Por este motivo no se ha podido realizar la reposición de los cierres laterales y el dintel del cuerpo de las compuertas bureau, y sustitución del metal babitt del cierre del umbral donde se deberá actuar cuando se realice la adecuación de las compuertas de seguridad.

Nuevo puente grúa.

Cámara desagüe fondo (Guadalest).



Detalle de las fugas del desagüe de fondo (Beniarrés).

Detalle de las fugas del desagüe de fondo (Beniarrés).

Los trabajos en la compuerta de regulación de dicho desagüe se tuvieron que realizar mediante buzos, através de dicho conducto, para así proceder al izado del tablero de la compuerta una vez asegurada

3.4.3 *Toma de Beniarrés*

Se han sustituido las cuatro válvulas de la toma que se encuentran en muy mal estado por oxidación, además la cámara se ha debido ampliar puesto que de dimensiones escasas para realizar cualquier maniobra.

Válvulas de compuerta en estado precario.

Detalle del ensanchamiento en la galería de la Toma de Beniarrés.

Los trabajos han consistido en ampliar la cámara mediante picado con martillo compresor hasta las dimensiones fijadas.

Por otra parte se ha debido sanear el conducto de toma aguas arriba, hasta localizar el conducto en buen estado, soldando nuevamente un tramo y continuando con las piezas y valvulería necesarias para la toma.

Cabe citar que a efectos de evaluar la posible repercusión de esta ampliación sobre el estado tensional en el cuerpo de presa en el entorno de la cámara, se contemplaba en el proyecto esta situación, suponiendo un 0,21% la variación sobre la sección de la presa.

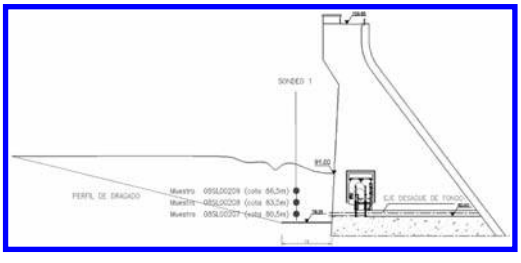
4 CARACTERIZACIÓN DE LODOS Y PROYECTOS DE DRAGADOS

La renovación de los equipos hidromecánicos de aguas arriba, que conforman los desagües de fondo de los embalses de Forata, Amadorio, Guadalest y Beniarrés, requieren del dragado previo de los sedimentos presentes en el entorno de

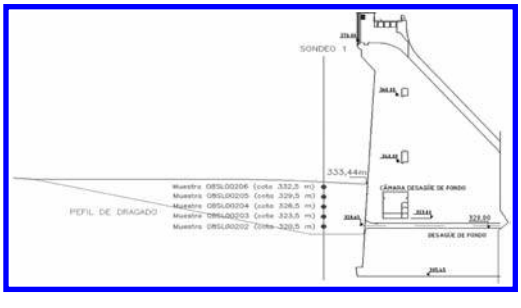
los mismos y que por la acumulación histórica han cubierto estos desagües hasta dejarlos literalmente enterrados.

Al objeto de plantear la solución constructiva más conveniente para este dragado y destino de los sedimentos, se ha realizado una toma de muestras de los sedimentos y un completo análisis posterior de los mismos, caracterizándose como residuos no peligrosos: código 17 05 06 "Lodos de drenaje distintos de los especificados en el código 17 05 05"(el 17 05 05 "Lodos de drenaje que contienen sustancias peligrosas"), en aplicación de la Orden MAM/304/2002. Por tanto, el destino final de los fangos dragados podrá ser un vertedero autorizado con esta codificación.

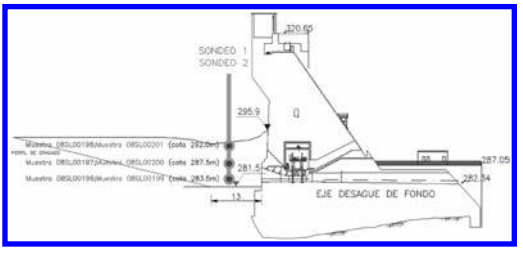
Por otro lado, el dragado se plantea mediante el empleo de bombas de succión y tuberías de transporte hasta playas de secado en las proximidades del vaso, como solución de menor impacto ambiental y mayores garantías de control de la calidad del agua en el embalse y aplicación de medidas preventivas.



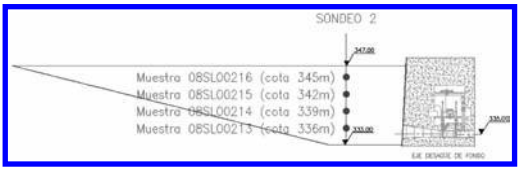
Perfil de dragado del embalse de Amadorio.



Perfil de dragado del embalse de Guadalest.



Perfil de dragado del embalse de Beniarrés.



Perfil de dragado del embalse de Forata.

4.1 *Batimetrías*

Mediante batimetrías con ecosonda y posicionamiento XYZ de los puntos de medición, se calculó la potencia de lodos acumulados junto al cuerpo de cada presa, mediante la batimetría de la ocupación actual de los mismos y la cartografía originaria del proyecto constructivo.

Para obtener un curvado más preciso se han densificado considerablemente la malla de puntos en toda la zona adyacente a la presa y se han hecho en la medida de lo posible las pasadas de la embarcación en paralelo al paramento de la presa, para conseguir una uniformidad en la toma de datos.

Con la realización de la batimetría pudo comprobarse la potencia y disposición de los sedimentos, observándose que en el embalse de Beniarrés presentaban un talud hacia el desagüe de fondo, lo que posibilitaba la toma de muestras en las proximidades del mismo y a diferentes profundidades mediante medios subacuáticos convencionales.

4.2 *Toma de muestras*

En los embalses donde los lodos cubren con grandes potencias los desagües de fondo. Se considera

EMBALSE	PROVINCIA	CAUCE	AÑO CONSTRUCCIÓN	CAPACIDAD Hm^3	USO	POTENCIA SEDIMENTOS
FORATA	Valencia	Magro	1.969	37,10	Riego	12,00
BENIARRÉS	Alicante	Serpis	1.962	27,00	Riego	11,25
GUADALEST	Alicante	Guadalest	1.964	13,00	Abastecimiento y riego	15,00
AMADORIO	Alicante	Amadorio	1.960	15,83	Abastecimiento	9,00

necesario el empleo de medios mecánicos como manuales para la toma de muestras.

Se ha realizado la toma de muestras mediante equipos de sondeo a rotación con recuperación de testigo a diferentes profundidades en el lodo en función de su potencia y muestras manuales mediante equipos de buceo profesionales y especialistas en trabajos subacuáticos en Beniarrés.

4.3 *Estudio de la cuenca y el vertido potencial al embalse*

La aportación de sedimentos a los embalses depende de la conservación de los suelos y de la erosión de la cuenca. Por un lado, la naturaleza y el origen de los sedimentos en los embalses proviene de los lechos de los ríos aguas arriba del embalse, puesta eventualmente en movimiento por los caudales mayores y de forma más continua a base de los diferentes vertidos por las poblaciones e industrias. Por otro lado, la fracción más importante procede de la erosión en las laderas de la cuenca vertiente a los mismos.

Se han estudiado las ocupaciones y actividades presentes en las cuencas vertientes a los embalses a fin de componer el escenario de potencialidad en los sedimentos, información ésta complementaria a los trabajos de ensayo en laboratorio como se verá más adelante.

4.4 *Ensayos realizados sobre los sedimentos*

En función de la información expuesta en el anterior apartado y de los datos de proceso, con arreglo a la normativa especificada por el RD 833/88 y RD 952/97 que lo modifica y el RD 363/95 sobre método de caracterización, los sedimentos objeto de estudio pueden ser encuadrados en el tipo nº 23, "lodos de dragado", del apartado B de la tabla 3 (Anejo I del RD 952/97).

Por tanto, para que estos lodos se caractericen como residuo tóxico y peligroso deben contener algún constituyente recogido en la tabla 4 de dicho Anejo, códigos C, y presentar simultáneamente alguna característica enunciada en la tabla 5 del Anejo, códigos H.

El enfoque del análisis efectuado ha sido la realización secuencial de los ensayos de características de peligrosidad, comenzando con los más sencillos (físico-químicos) y terminando con los más sofisticados.

En conclusión, para cada muestra de lodo se definirá su clasificación, que podrá ser:

- *Residuo peligroso, que sólo admite su gestión por gestor autorizado*.
- *Residuo no peligroso, que podrá trasladarse a vertedero con autorización para este tipo de sedimento*.

4.4 *Determinación del procedimiento de dragado*

Una vez comprobada la ausencia de toxicidad de los sedimentos, se han estudiado diferentes alternativas para el dragado de los lodos en el entorno de los desagües de fondo, su tratamiento posterior y su destino final.

Los volúmenes de dragado de fango seco resultantes son los siguientes:

- Forata: 25.000 m^3
- Amadorio: 30.000 m^3
- Guadalest: 38.000 m^3
- Beniarrés: 25.000 m^3

La elección del equipo de dragado más adecuado para un determinado trabajo se realiza teniendo en cuenta una serie de aspectos siendo los principales:

- Las características del fango a dragar.
- La profundidad de dragado.
- La cota de agua del embalse.
- Las condiciones de secado y trasporte.
- Las condiciones ambientales.
- La calidad del agua del embalse.

Se ha seleccionado la alternativa de un equipo de dragado compuesto por:

- Pontona modular con bombas de succión de fangos de 5000 hp o draga fluvial.
- Tuberías de succión y transporte de fangos con cúter de corte en la admisión.

Con estos equipos, los rendimientos son:

- Volumen de dragado: 200 m^3/día, con una mezcla de 15% sedimento y 85% agua.

5 CONCLUSIONES

La renovación de los equipos hidromecánicos en tomas y desagues de los embalses de Forata, Amadorio, Guadalest y Beniarrés, han sido acometidos en dos fases atendiendo la primera de estas a elementos singulares con soluciones, que requieren medios y procedimientos en algún caso de cierta complejidad, sin necesidad de realizar operaciones de dragado.

La sustitución de las compuertas de los DF en contacto directo con el agua, requieren del dragado previo de los sedimentos presentes en el entorno de los mismos y que por la acumulación histórica han cubierto estos desagües hasta dejarlos literalmente enterrados.

Al objeto de plantear la solución constructiva más conveniente para este dragado y destino de los sedimentos, se ha realizado una toma de muestras de los sedimentos y un completo análisis posterior de los mismos, caracterizándose como

residuos no peligrosos: código 17 05 06 “Lodos de drenaje distintos de los especificados en el código 17 05 05”(el 17 05 05 “Lodos de drenaje que contienen sustancias peligrosas”), en aplicación de la Orden MAM/304/2002. Por tanto, el destino final de los fangos dragados podrá ser un vertedero autorizado con esta codificación.

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Rehabilitación de los desagües de fondo de la Presa de Susqueda

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RESUMEN: La rehabilitación de los desagües de fondo de la Presa de Susqueda, además de tener que superar las dificultades que impone su particular geometría, ha sido proyectada de modo que modernice unos desagües diseñados bajo unos criterios que hoy no serían admisibles.

El difícil acceso a los desagües, tanto aguas arriba con casi 90 metros de columna de agua, como aguas abajo con 30 metros de altura sobre el cuenco amortiguador y 90 metros de caída desde coronación obligó, a utilizar métodos y estructuras especiales para el sellado del conducto, el desmontaje de los sistemas hidromecánicos existentes y el montaje de los nuevos.

Actualmente, los desagües de fondo de la presa constan de una válvula mariposa de seguridad y de una válvula howell bunger de regulación. Esta configuración tiene importantes deficiencias funcionales, como los problemas de cavitación y obstaculización producidas por el obturador de la válvula mariposa, o los radios del cono de la válvula howell bunger.

La nueva configuración de los desagües ha sido proyectada utilizando válvulas de asiento plano y paso circular®, que permiten obturar el conducto con aguas vivas, disminuyendo ostensiblemente los problemas de cavitaciones, y dejando totalmente libre el paso del agua en aperturas totales. La disipación de la energía del chorro, antes a cargo del cono de la válvula howell bunger, se realizará mediante una salida cónica fija.

Este trabajo engloba los principales aspectos involucrados en la rehabilitación, tanto constructivos como hidromecánicos y funcionales.

ABSTRACT: Renovation of the bottom outlets of the Susqueda dam, in addition to having to overcome the difficulties imposed by its particular geometry, was also planned in such a way that modernises some outlets designed under criteria that today would not be entirely acceptable.

Difficult access to the outlets, both upstream with almost a 90 m column of water, and downstream with a height of 30 m above the stilling basin and a 90 metre fall from the top of the dam, forced special methods and structures to be used for sealing the conduit, for dismantling the existing hydro-mechanical systems and for the assembly of the new ones.

Currently the dams high-pressure outlets have a butterfly safety valve and a Howell-Bunger valve for regulation. This configuration has significant functional defects, such as cavitation and obstruction problems caused by the seal on the butterfly valve or the radii of the cone on the Howell-Bunger valve.

The new configuration for the outlets has been planned using flat seat and circular flow valves which enable the conduit to be sealed by tidal waters, visibly reducing the cavitation problems and leaving the flow of water totally free when fully open. The dissipation of the streams energy, which before was controlled by the Howell-Bunger valve, will be done using a fixed conical outlet.

This work covers the main aspects involved in the renovation, both construction as well as hydro-mechanical and functional aspects.

1 INTRODUCCIÓN

La presa de Susqueda, terminada de construir en 1968, forma parte del aprovechamiento hidro-eléctrico integral del tramo inferior del río Ter. Es utilizada primordialmente para el abastecimiento de agua a Girona y Barcelona junto con la producción de energía eléctrica.

Es una presa de tipo bóveda simétrica de doble curvatura, con 128 m sobre cimientos, un ancho de coronación de 5 m, y una longitud total, incluyendo los estribos, de 500 m. Los desagües de fondo están

a 37 m desde el cuenco y a 90 m desde coronación (Rebollo 1967). La Figura 1 muestra un esquema de la presa.

Los desagües de fondo están constituídos por una entrada abocinada con una reja de gruesos y un alojamiento para una ataguía, continúa una conducción recta de 1500 mm de diámetro y unos 12 m de longitud, una válvula mariposa de 1800 mm como elemento de seguridad, y una válvula Howell Bunger de 1300 mm de regulación. En la Figura 2 se muestra un esquema de estos desagües.

Desde la construcción de la presa se ha producido el natural deterioro de los elementos hidro-mecánicos. El difícil acceso a estos elementos impide realizar un adecuado mantenimiento, disminuyendo las garantías de seguridad y funcionamiento. A estas deficiencias se debe sumar el hecho de que las válvulas de mariposa, si bien son elementos de seguridad muy fiables, están diseñadas para trabajar en conductos a bajas velocidades; de no ser así, los problemas de vibraciones y cavitaciones pueden ser muy significativos (CNEGP 1997).

Con el objetivo de subsanar estas deficiencias se ha acometido el *Proyecto de renovación de los desagües de fondo de la presa de Susqueda*.

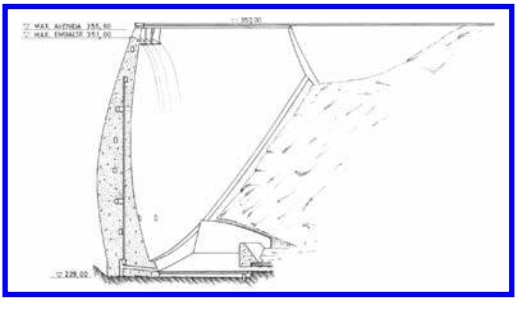

Figura 1. Esquema de la presa de Susqueda.

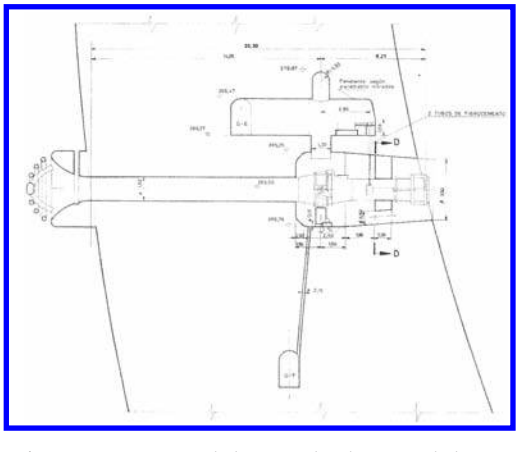

Figura 2. Esquema de los actuales desagües de la presa de Susqueda.

2 NUEVOS DESAGÜES

La solución adoptada para la renovación de los desagües de fondo está fuertemente marcada por las características geométricas de toda la presa.

La renovación de los conductos consistirá en reemplazar los elementos existentes por los siguientes (desde aguas arriba):

- Cono de reducción de 1500 (mm) a 1400 (mm) para empalmar el tubo existente con las nuevas válvulas.
- Válvula de seguridad de compuerta de asiento plano y paso circular® de diámetro 1400 (mm) con aireación mediante ventosa.
- Válvula de regulación de compuerta de asiento plano y paso circular® de diámetro 1400 (mm) con aireación atmosférica.
- Carrete de boca de hombre.
- Cono de reducción entre carrete de boca de hombre y disipador de energía de 1400 (mm) a 1300 (mm).
- Disipador de energía.

2.1 *Válvulas de seguridad y regulación*

Son del tipo compuerta de paso circular y asiento plan[P]. Este tipo de válvulas, estando abiertas, no oponen resistencia al paso de elementos en el conducto, y el diseño de asiento plano impide la acumulación de sedimentos. Están concebidas para sustituir a las válvulas de tipo *Bureau*; conservan el cierre inferior plano como estas, eliminando la ranura inferior de las válvulas compuerta circulares convencionales y conservando el paso circular. De esta forma se eliminan las transiciones de sección circular a rectangular, necesarias aguas arriba y abajo de las válvulas *Bureau*, mejorando la hidráulica de la conducción y acortando la longitud de montaje (Marín and García 2002), que en cámaras cortas como las de Susqueda estas necesidades de distancias pueden ser críticas. La Figura 3 muestra una válvula de este tipo.

Las válvulas serán libres de mantenimiento y estarán semi-hormigonadas hasta debajo de la brida del cuerpo superior, de modo que se permita el desmontaje, permitiendo la auscultación del obturador y del interior de la válvula, así como la sustitución del cilindro de accionamiento. Las válvulas serán autoresistentes, es decir, podrán resistir los esfuerzos hidráulicos sin la colaboración del hormigón.

Están calculadas para una presión de trabajo de 100 mca con un coeficiente de seguridad que incluye los posibles efectos dinámicos. Toda la válvula trabajará en todos los casos en tensiones sobradamente por debajo del límite elástico, asegurando así que las deformaciones no sean excesivas,

dotándola de la serviciabilidad requerida en estos elementos de obturación.

Disponen de tomas de aire perimetrales que airean adecuadamente el chorro de agua, estabilizándolo y disminuyendo las vibraciones.

2.2 *Carrete con boca de hombre*

Su principal función es unir la válvula de regulación con el disipador de energía. Cuenta con una boca de hombre que permite la entrada al interior del conducto desde la cámara de válvulas para la revisión, montaje y desmontaje de la válvula, como se muestra en la Figura 4.

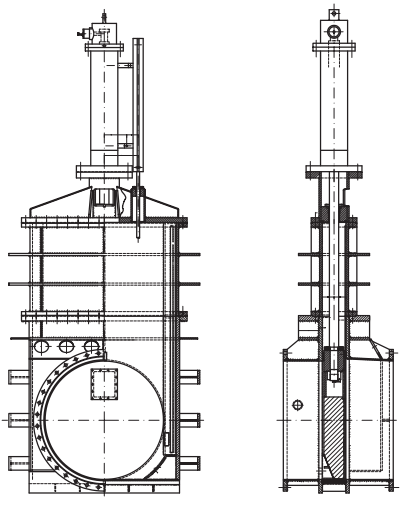

Figura 3. Esquema de una válvula de paso circular y asiento plan®.

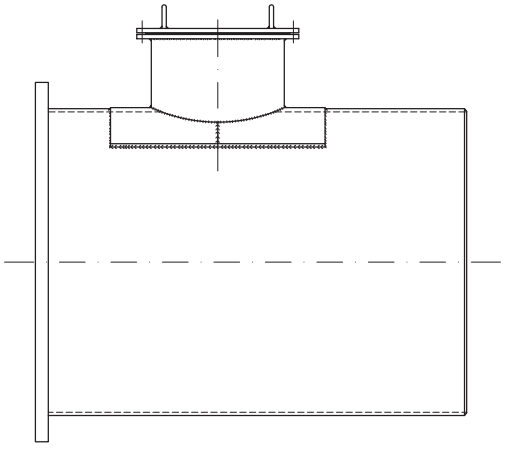

Figura 4. Esquema del carrete de boca de hombre del nuevo desagüe.

2.3 *Disipador de energía*

Cuenta, en la salida, con un cono fijo sujeto al cuerpo del carrete mediante dos cartelas verticales interiores a modo del cono de una válvula Howell Bunger (Figura 5), y refuerzos exteriores radiales en las otras direcciones. Esta disposición aporta la resistencia necesaria al cono, e impide que el chorro de agua no estabilizado, proveniente de la válvula de regulación en aperturas parciales, choque contra los nervios interiores produciendo vibraciones. De este modo, se emula el comportamiento hidráulico del desagüe antiguo al evacuar al cuenco.

2.4 *Métodos de construcción*

Para llevar a buen término la solución antes descrita, es necesario disponer de unos medios que permitan realizar los trabajos de forma eficaz y segura, lo que genera la necesidad de:

- Sellar el conducto usando la recata de alojamiento de la ataguía, cuyas dimensiones deben ser diseñadas a partir de los antecedentes documentales y de unas inspecciones mediante buzos, para confirmar la geometría de su alojamiento.
- Disponer de un sistema que permita transportar elementos entre el desagüe y la coronación.
- Disponer de una superficie de trabajo que permita cargar/descargar el pórtico con seguridad y facilite los trabajos de desguace/montaje del conducto.

Debe tenerse en cuenta que de la solución adoptada depende el éxito de los trabajos, por lo que se han incluido las siguientes premisas de diseño:

- *Ataguía:* Chapón macizo de acero cuya función principal es impedir el paso de agua a través de los conductos de desagüe de fondo, proporcionando de este modo seguridad y facilidad de realización de los trabajos en seco, es decir, sólo con las fugas propias de estos sistemas. La estanquidad se logra por aplastamiento de una junta tórica. Consta de un sistema de válvulas que permite el equilibrado del conducto (Figura 6).
- *Pórtico en coronación:* Diseñado para maniobrar piezas de hasta 5000 kg. Tendrá tres niveles

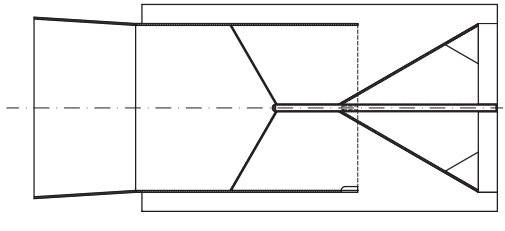

Figura 5. Esquema del disipador de energía del nuevo desagüe.

de seguridad independientes entre sí respecto al vuelco. Estos sistemas de seguridad son el contrapeso propio del pórtico, el atirantado de las cuatro patas del pórtico y el anclaje de las vigas de repartición a la losa de coronación (Figura 7).

- *Plataforma de trabajo:* Estructura de acero reticulada, anclada mediante placas al intradós de la presa (Figura 8). Diseñada para soportar un colapso parcial de la estructura de soporte. Las vigas, articuladas en su unión a las placas de anclaje, disminuyen el momento aplicado sobre el hormigón. Consta de una parrilla de repartición que distribuye los esfuerzos en toda la estructura. Además de los soportes rígidos, la plataforma está soportada por dos tensores laterales que sujetan una viga maestra capaz de sostener la plataforma. Debido a la curvatura de la presa se han rasgado los taladros de las uniones de vigas, de modo que la plataforma se adapte a esta curvatura.

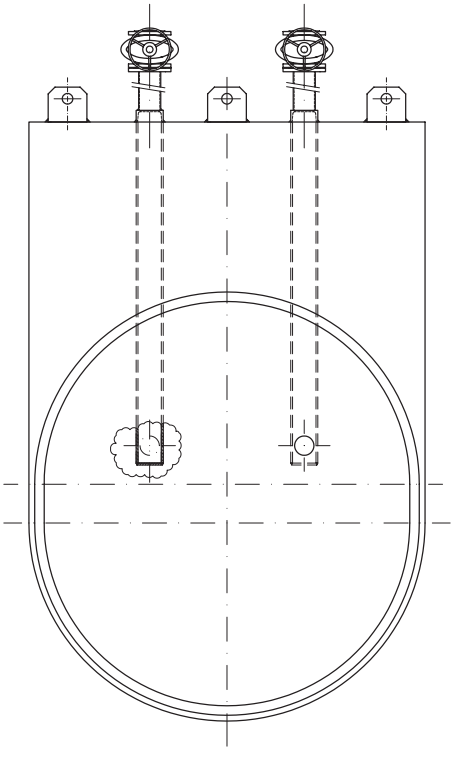

Figura 6. Esquema de la ataguía de los desagües de la presa de Susqueda.

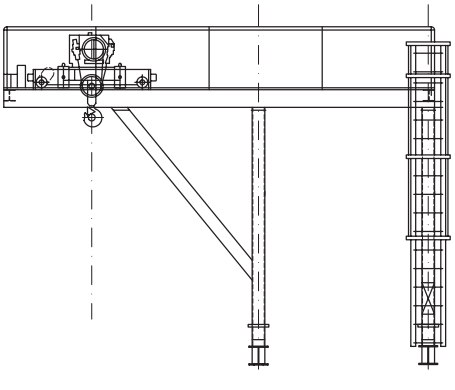

Figura 7. Esquema del pórtico de coronación de la presa de Susqueda.

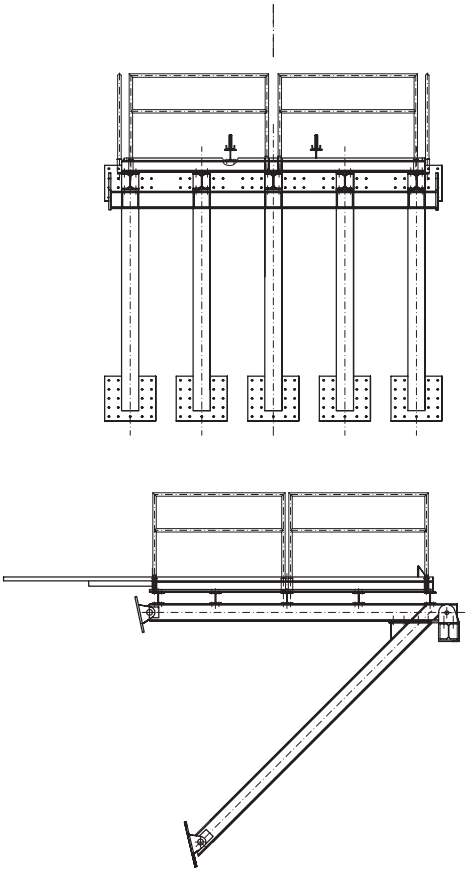

Figura 8. Esquema de la plataforma de intradós de la presa de Susqueda.

3 FASES CONSTRUCTIVAS

Con la definición de la solución y de los elementos auxiliares adoptados, las fases constructivas quedan dadas por:

- Inspección sub-acuática de la recata de ataguía.

- Montaje de pórtico y colocación en su posición de trabajo. Incluso pruebas de carga con dado de hormigón de 5000 (kg).
- Colocación de la ataguía en su recata.
- Montaje de plataforma. Incluso pruebas de carga de la plataforma con dado de hormigón de 5000 kg.
- Desguace de válvula Howell Bunger.
- Demolición de anillo de hormigón.
- Desguace de carretes y válvula mariposa, y otros elementos en la cámara de válvulas.
- Demolición de losa de hormigón, y abertura de agujeros cilíndricos para aducciones atmosféricas de aire.
- Montaje de válvulas y carretes.
- Hormigonado de cámara de válvulas.
- Montaje de aducciones atmosféricas de aire y de ventosa.
- Instalación óleo-hidráulica y eléctrica.
- Desmontaje de plataforma.

4 CONCLUSIONES

Se ha presentado una solución de renovación de los desagües de fondo de la Presa de Susqueda. Se han descrito las principales características hidromecánicas de la solución, así como el método constructivo y los elementos auxiliares mediante los cuales se ha llevado a cabo.

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Actuaciones de mejora en la presa de Arquillo de San Blas

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RESUMEN: La presa de Arquillo de San Blas, fue construida en 1962 sobre el río Guadalaviar o Turia. Se muestra en esta comunicación el proceso seguido para la renovación total de las instalaciones del túnel de desvío así como algunos elementos de los desagües de fondo y toma de riego. Se pretende incidir sobre el acondicionamiento del túnel en el que se ha dragado la boca de acceso así como el propio túnel hasta la cámara donde se albergan las compuertas.

ABSTRACT: Arquillo dam of San Blas, as built in 1962 on the Guadalaviar´s River. Shown in this paper the process followed for the complete renovation of facilities in the tailrace tunnel and some elements of lower outlet and making irrigation. It is intended to influence the preparation of the tunnel that has dredged the manhole and the tunnel itself to the chamber where the gates are located.

1 INTRODUCCIÓN

En la actualidad se presenta una tendencia mundial encaminada a la conservación y mantenimiento del parque presístico, adaptándolas a las nuevas leyes y códigos que deben ser cumplidos.

Existen hoy nuevos condicionantes que ponen de manifiesto lo comentado anteriormente, Charlwood_et_al. G.Membrillera_et_al.

- El envejecimiento del parque de presas (la mayoría de las estructuras superan los treinta años de antigüedad y un porcentaje muy amplio los cincuenta años en explotación).
- La demanda de mayores niveles de seguridad para la población y los bienes ubicados aguas debajo de las presas.
- Un mayor escrutinio de la financiación de los programas de seguridad de presas.
- La creciente dificultad para acometer la construcción de nuevas estructuras debido a previsiones sociales y medioambientales.
- La necesidad de optimizar la gestión de sistemas de recursos hídricos y aumentar la capacidad de regulación de los mismos con objeto de dar respuesta a una demanda creciente de abastecimiento; todo ello en un contexto de aparente incremento de eventos climatológicos extremos (avenidas y sequías).
- Una mayor énfasis hoy en día en el mantenimiento, la seguridad, la reparación y la extensión de la vida útil de las presas, más que en su construcción.

A continuación se presenta una actuación que se enmarca en la casuística expuesta, renovación de las instalaciones del túnel de desvío y acondicionamiento del desagüe de fondo y la toma de abastecimiento, en la Presa del Arquillo de San Blas (Teruel), sobre el río Guadalaviar.

1.1 *La presa de Arquillo de San Blas (Teruel)*

La presa de Arquillo de San Blas, se construyó en el año 1962, sobre el cauce del río Guadalaviar o Turia.

La presa es de tipo gravedad, de hormigón en masa de planta recta, con una altura máxima sobre cimientos de 54 m. La capacidad de la misma es de 21 Hm^3.

Esta presa fue concebida fundamentalmente para la defensa contra avenidas y para regulación

Figura 1. Vista presa del Arquillo.

del río para su aprovechamiento en el regadío y abastecimiento de agua a la Ciudad de Teruel.

El aliviadero es en lámina libre, con una anchura de vertido de 50 m, en cuatro vanos de 12,5 m y capacidad prevista para evacuar 500 m^3/s.

La presa cuenta con una toma de riego en la margen derecha tanto para el riego, como para el abastecimiento de Teruel, un desagüe de fondo (con dos conductos) en la zona central y un túnel de desvío que discurre por la margen izquierda.

Las edificaciones para la explotación de la presa se ubican en la ladera izquierda del valle, por encima de la cota de coronación de la presa.

2 ACTUACIONES PREVISTAS

2.1 *Toma de agua*

La toma de agua dispone de una compuerta rectangular tipo Bureau de 1200 × 1400 mm de guarda y que es accionada por un mecanismo de crick de aceite y de una válvula de regulación tipo Howell Bunger de Ø 1,40 m con un caudal máximo de 24,4 m^3/seg.

Está válvula se colocó en 1973, en sustitución de una válvula Larner Johnson, que debido a la composición salitrosa del agua del embalse, producía frecuentes averías en la válvula.

Debido a la dificultad de controlar los caudales de trabajo, es decir mantenimiento del caudal ecológico y para el riego (que no suelen superar los 2 m^3/seg), ha hecho aconsejable la colocación de una válvula que permita aperturas adecuadas a estos caudales.

Se ha previsto el montaje de una válvula Howell Bunger de 400 mm de diámetro con carrete deflector incorporado junto a la existente.

2.2 *Desagüe de fondo*

Consta de dos conductos de sección circular de 1500 mm de diámetro y 34,02 m de longitud, con dos compuertas Bureau cada uno de 1500 × 1200 mm y una capacidad máxima de desagüe de 60,4 m^3/seg, ambas accionadas por un mecanismo de crick de aceite.

En esta zona se prevé la renovación de los conductos que constituyen el by-pass, aumentar la capacidad del sistema de aireación de las compuertas Bureau y la colocación de un pupitre electrohidráulico para el mando de las cuatro compuertas Bureau.

2.3 *Túnel de desvío*

Consta de un único conducto con dos válvulas Bureau en serie de 1500 × 1200 mm, ambas con accionamiento por mecanismo de crick en aceite y con una capacidad máxima de desagüe de 32 m^3/seg.

El túnel de desvío data de los años 60 sin que hasta la fecha se haya llevado a cabo ninguna actuación encaminada a mejorarla, únicamente las tareas de mantenimiento y conservación y reparaciones puntuales motivadas por averías.

En la actualidad se encuentra fuera de servicio, porque el túnel de desvío desde la embocadura hasta el tablero de la compuerta Bureau de guarda, se encuentran totalmente obturados por lodos.

Las obras proyectadas en esta zona comprenden el dragado de lodos, sustitución de los tableros de las compuertas Bureau, campanas, cilindros, la instalación eléctrica y oleohidráulica, así como la renovación del by-pass.

3 ESTADO ACTUAL DE LAS OBRAS

3.1 *Túnel de desvío*

Esta operación es necesaria para extraer los lodos en el interior del túnel así como en la embocadura del mismo. El volumen aproximado total de ambos asciende a unos 3.500 m^3.

Se han utilizado dos procedimientos de dragado, uno para el dragado de la embocadura y exterior del túnel de desvío y otro para el dragado del interior del mismo.

Para el dragado del exterior del túnel se ha utilizado una bomba Mamut y para el dragado interior un carro de dragado.

A continuación se definen los dos sistemas:

3.1.1 *Dragado de la embocadura y exterior del túnel*

Este se ha realizado mediante una bomba tipo Mamut con vertido a un contenedor metálico (cántara) situado en la superficie. Este recipiente metálico de 6,00 × 2,00 × 1,50 recibe los lodos bombeados desde el fondo y allí se realiza la separación de los lodos y de los restos vegetales que puedan ser succionados por la Mamut.

Después de decantar los materiales no bombeables, los lodos restantes serán bombeados hasta la zona de vertido mediante un grupo de 2 bombas sumergibles, cuya capacidad individual es mayor que el caudal aportado por la Mamut. De esta manera se consigue que no pueda rebosar el depósito decantador por exceso de dragado.

La bomba Mamut consiste en dos tramos de tubo de polietileno de diámetro 500 mm, unidos con un codo. El tramo de la embocadura (Figura 2) succiona el material. El proceso seguido es la disgregación del material sedimentado en el fondo y la succión del mismo. Para ello la bomba Mamut monta en su cabezal de toma (2.000 kg), que es dirigido desde la superficie mediante cables metálicos que sitúan en cada momento el cabezal en la zona deseada, un disgregador de fango por agua a presión (200 bar) que actúa girando (1 rpm) alrededor de su eje, y que produce un corte de los sedimentos de 1 m de radio con el cabezal de corte (Figura 3).

La succión del material se produce inyectando, en la embocadura, mediante un compresor, aire a la presión necesaria para que la columna de Aire + Lodo + Agua que se forma en el interior del tubo tenga una densidad menor que la del agua y por lo tanto ascienda hasta la cántara de vertido. El tramo de la desembocadura vierte el material a la cántara (Figura 4) que hay en la pontona en superficie. En la cántara se dispone de una bomba que impulsa el material dragado a una zona de vertido, aislada del embalse mediante unas barreras de contención y antitubidez (Figura 5).

El dragado se programa por terrazas de 1 a 2 m de profundidad. Los restos vegetales y escombros de difícil extracción con la bomba Mamut, se extraerán mediante la grúa instalada usando una bivalva o pulpo según sea su naturaleza.

3.1.2 *Dragado de lodos y fangos en el interior del túnel*

Dada la peligrosidad que entraña el dragado en el interior del túnel para los buzos junto con los bajos rendimientos así como la nula, se ha ideado un sistema de dragado que no requiere la entrada de buzos en el túnel salvo en algún momento puntual. A continuación, se pasa a describir dicho.

Figura 2. Embocadura Bomba Mamut.

Figura 3. Cabezal de corte.

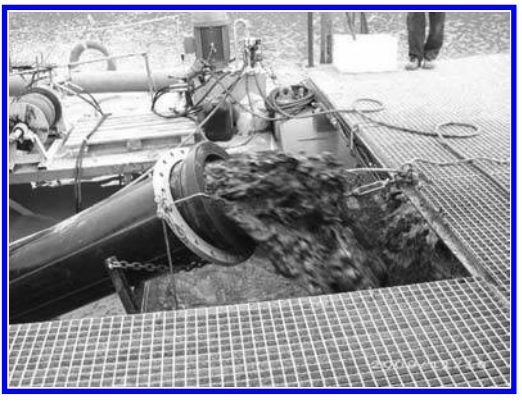

Figura 4. Vertido de los lodos a la cántara.

Figura 5. Barreras antiturbidez.

El túnel de desvió es sensiblemente horizontal y el método de dragado difiere en cuanto a que el cabezal de corte va montado sobre una maquina de orugas metálicas de unos 1.500 Kg. de peso, esta máquina es dirigida desde el centro de control en superficie.

La máquina de orugas es accionada hidráulicamente y permite los movimientos de giro, avance y retroceso. En este dragado interior, la aspiración de los lodos removidos, se realiza mediante el mismo sistema de succión que la bomba mamut.

El carro de dragado está formado por un tubo horizontal, apoyado sobre dos líneas de orugas que permiten al tubo desplazarse sobre una superficie irregular. Hay un tramo aproximadamente vertical por donde se eleva el material dragado y un tercer tubo que lo deposita en la cántara. Los tramos se conectan con los correspondientes codos. El sistema de succión es también mediante aire a presión en la embocadura, y el material, se disgrega mediante el mismo cabezal de corte que el mencionado anteriormente.

En la actualidad, con la ayuda de expertos se está reestudiando el sistema previsto en proyecto para colocar un sistema que permita la estanqueidad en la cámara de compuertas, donde se pretende actuar.

Una vez limpio, analizada la geometría, así como los materiales y su calidad que conforman el túnel, todavía no se ha decidido cual va a ser la metodología a seguir para realizar la sustitución de los tableros de las compuertas. La idea de colocación únicamente de un tapón en el túnel de desvío para conseguir la estanqueidad aguas abajo del mismo, no parece suficiente debiéndose completado con algún otro sistema.

3.1.3 *Sistema de aireación del túnel de desvío*

El túnel de desvío tiene una longitud de 177 m aproximadamente desde la compuerta de aguas abajo hasta su conexión con el río Guadalaviar.

Presenta una geometría en planta caracterizada por dos tramos rectos unidos por una curva circular de radio de 9 m y un desarrollo de 90 grados.

La sección es una bóveda formada por tramos circulares de radio 3 m en la base y hastiales. Se puede asemejar a una sección circular de 3 m de diámetro.

En alzado presenta una pendiente constante de 0.005864 m/m.

Debido a la particularidad de este túnel, sobre todo a la curva de 90 grados y radio de 9 m se ha tratado de estudiar el efecto de la aceleración transversal sobre la lámina de agua. Para ello se ha modelizado mediante un programa de elementos finitos (GUAD 2 D) y se observa un desplazamiento lateral de la lámina de agua (Figura 6). Este efecto unido a que se producen velocidades muy elevadas,

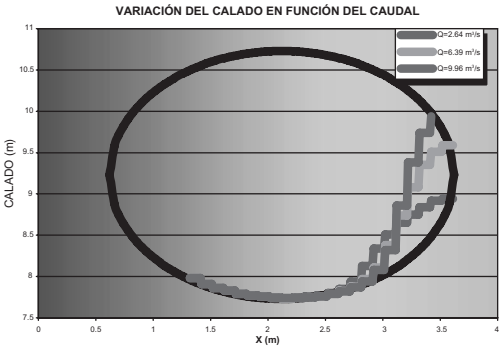

Figura 6. Variación del calado en función del caudal.

del orden de 10 a 15 m/s y ante la dificultad de que el modelo no puede representar la posible entrada en presión se considera necesario construir un modelo físico que represente los fenómenos que se producen y se pueda fijar de esta forma las normas de explotación de este túnel. La realización de este modelo ya ha sido solicitado por parte de la CHJ al CEDEX.

Así una vez conocido el funcionamiento del túnel se procederá a diseñar el sistema de aireación del mismo.

3.2 *Resto de actuaciones en túnel de desvío*

Para la mejora de la aireación, inicialmente venía planteado demoler una parte del hormigón de aguas abajo de la compuerta Bureau de regulación y colocar una caja de vientos, con tres conductos de aireación de 400 mm de diámetro, con válvulas aductoras de aire que comunica con las perforaciones realizadas en la parte superior del conducto, mejorando de esta forma la aireación que actualmente ha sido considerada escasa.

Tras el estudio de esta actuación, se vio la particularidad que presenta éste túnel.

Inmediatamente aguas abajo de la compuerta, se presenta un aumento de sección sin transición (se pasa del orificio de la compuerta 1200×1500 mm a un arco bóveda de 3000 mm de diámetro aproximadamente) y además presenta un trazado en planta con una curva de 90 grados y de radio de giro 9 m.

Por todo lo anterior se decidió cambiar la aireación proyectada y realizar una perforación en la cámara del túnel de desvío, de 700 mm de diámetro, de forma que entre el aire directamente al túnel, aguas abajo de la compuerta Bureau de regulación. Así como realizar la aireación directa de ambas compuertas, de regulación y de seguridad mediante conductos de 150 mm.

Para finalizar con las obras del túnel se prevé la revisión de los tableros y su posible renovación, dependiendo de su estado.

Para ello sería necesario la colocación de un elemento, tapón o escudo, que corte el agua para trabajar en el interior del túnel.

En este momento, todavía no se ha podido acceder al túnel y por lo tanto no se sabe si será necesario la colocación del escudo.

3.3 *Toma de agua*

Para la colocación de la nueva Howell Bunger, se realizará un injerto en la tubería existente de 1400 mm de diámetro, un carrete acodado de 500 mm de diámetro. Tras la válvula de compuerta de acero inoxidable, se colocará la nueva Howell Bunger de 400 mm de diámetro con carrete deflector de chorro incorporado a la válvula.

En la cámara de la toma de agua se montará un pupitre electrohidráulico para el mando de la compuerta Bureau de guarda.

Para la aireación de la Bureau de guarda de la cámara de la toma de agua, se prevé colocar una ventosa bifuncional de 150 mm de diámetro con una válvula de compuerta manual.

3.4 *Desagüe de fondo*

Para la ejecución de las obras es necesario tapar la toma de agua del by-pass para así poder sustituir aquellos elementos que lo requieren. El método empleado para ello es mediante un tapón tronco-cónico.

Para la mejora de la aireación, inicialmente se había propuesto demoler una parte del hormigón de aguas abajo de la compuerta Bureau de regulación y colocar una caja de vientos, con dos conductos de aireación de 400 mm de diámetro, con válvulas aductoras de aire mejorando de esta forma la aireación.

Por ello aprovechando los conductos de 150 mm de diámetro, que dan la aireación actual directamente a la compuerta, colocar a partir de las bridas dos aductoras de este diámetro, esto se realizará tanto en la compuerta de guarda como en la de regulación. Para completar la aireación, se realizará aguas abajo de la compuerta Bureau de regulación, suficientemente alejada del tablero para no causarle ningún deterioro y aprovechando la ubicación del sumidero actual, este se perforará dándole un diámetro de 350 mm de diámetro para colocar una aductora de 300 mm. Independizando la aireación para cada conducto, que en la actualidad es común para los dos conductos.

Además, de estas obras se montará un pupitre electrohidráulico para el mando de las cuatro compuertas Bureau.

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Other related topics

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The impact of hydrology on the adequacy of existing dams to safety standards—the Namibian experience

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ABSTRACT: In an arid country like Namibia, floods generally represent the most important loading condition on a dam. Dam building in Southern Africa (including Namibia) went through a dam building boom period during the 1960's and 70's. Design of these dams (especially the hydrological design) was done up to 10 years before construction. As a result for the majority of cases, hydrology records of only between 10 and 30 years were available during design. During a safety assessment of Hardap Dam (an asphalt concrete faced rockfill dam with a gated spillway and the largest dam in Namibia) in 2006/07, it became evident that the lack of long enough hydrological records has a negative impact of its adequacy to present day safety standards. This case history will be discussed in detail by way of a sensitivity analysis indicating the importance of having a sufficient record length. The impact of not satisfying current safety standards will also be discussed as well as the proposed remedial measures.

1 INTRODUCTION

The large majority of dams in Namibia was designed and built during the 1950's, 60's and 70's. As a result for the majority of cases, hydrology records of only between 10 and 30 years were available during design as the detail monitoring of rivers in Namibia only started in the 1940's.

Hardap Dam (in the south of the country) is the largest dam in Namibia (total storage capacity of 294.593 million m^3) and the downstream irrigation scheme is one of the main food producing centers of Namibia.

On 24 to 26 February 2006 extensive damage occurred in the town of Mariental and to the irrigation scheme downstream of Hardap Dam. This was due to flooding in the Fish River following the opening of the sluice gates of the Hardap Dam during an extreme flood event in the Fish River.

A professional assessment of the flood situation (including a detail dam safety assessment) was carried out in 2007 under the auspices of the Ministry of Agriculture, Water and Forestry (MAWF). The purpose of the assessment was to propose measures to prevent a future flood disaster, or to mitigate as much as possible the consequences of an unavoidable flood. During the study it became evident that the lack of long enough hydrological records during the original design had a negative impact on the safety rating of the dam. (It was not adequate compared to present day safety standards).

2 HARDAP DAM

2.1 *General information*

Hardap Dam is located on the Fish River, some 17 km north north-west of the town Mariental in the southern part of Namibia (see Figure 1). The catchment is large (13 550 km^2) and the dam functions primarily as storage of water for irrigation, domestic and industrial use. The dam, designed and built under the auspices of the Department of Water Affairs of the then South West Africa by Dyckerhoff & Widmann (dam structures) and Strabag (asphalt concrete lining), was completed in 1963. It is currently owned and operated by the Namibia Water Corporation Ltd (NAMWATER).

The 35.9 m high dam wall is a composite structure consisting of a centrally located controlled ogee concrete gravity spillway with four radial gates. These are flanked by asphalt concrete faced

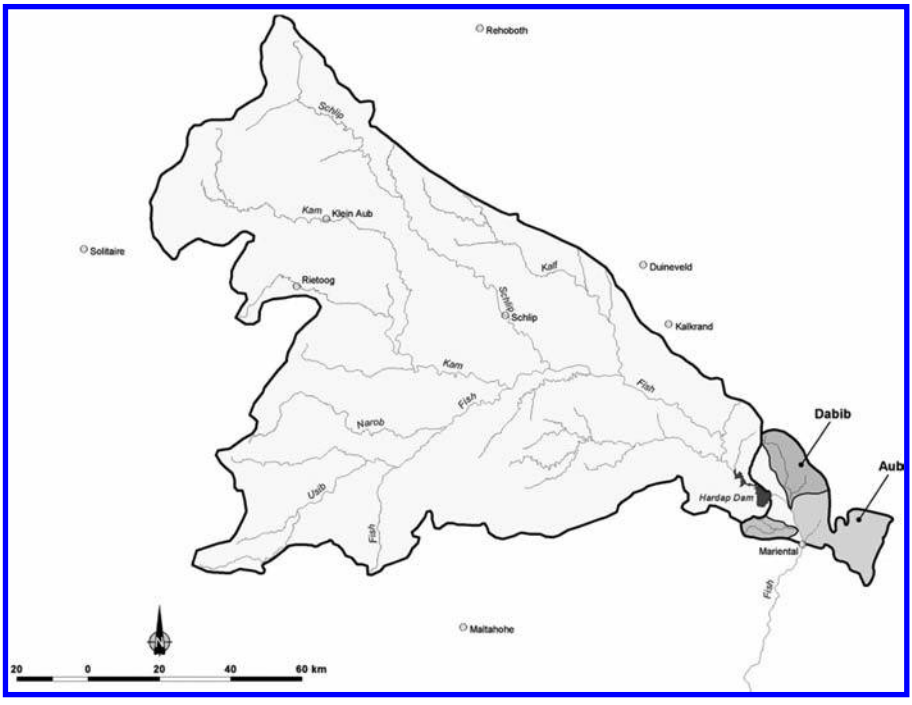

Figure 1. Location map of Hardap Dam.

rockfill embankments as well as one secondary embankment on the left flank approximately 1 km north north-west of the main embankment. Two saddle structures are located on the right bank approximately 120 m and 270 m respectively south east of the main embankment (see Figure 2). An auxiliary spillway is located on the outside of the secondary embankment on the left bank and a concrete apron is provided on the downstream toe of the spillway. The asphalt concrete facing was rehabilitated in 1996.

2.2 *Hydrology*

An evaluation of the flood hydrology of the Fish Basin catchment area upstream of Hardap Dam was carried out by Hattingh (2007a) as part of the assessment.

A comparison with the original hydrology used in the design showed a marked increase in the recommended flood peaks of the whole range of probabilities of exceedance. This can be attributed to the fact that the original design only used the results of the statistical analysis based on a record of 18 years whereas the results by Hattingh (2007a) are based on 65 years of continuous record. (A summary of the original design and current recommended representative flood peaks are given in Table 1).

It is important to note that in the original design a flood with a probability of exceedance of 0.1% was used as the extreme flood event (only 6 400 m^3/s). This does not comply with current internationally accepted criteria for a dam of the size of Hardap Dam and a recommendation to use a Safety Evaluation Floodpeak (SEF) of 11 640 m^3/s based on the SANCOLD (South African National Committee on Large Dams) guidelines (1991) was made by Hattingh (2007a). (The SEF per definition is the maximum extreme flood event for which the spillway system of a dam should be adequate). It is important to note that the SEF was determined empirically using a Franco-Rodier based methodology of historically observed flood peaks (Kovacs 1988). From the results of the statistical analysis it is evident that the probabilities of exceedance of the SEF peak is 0.035%.

The most important aspects relating to the evaluation by Hattingh (2007a) are as follows:

- The flood peaks were calculated using the results of the statistical analysis (General Extreme Value (GEV) distribution);
- A hydrograph based on observed historical hydrographs (with a time of concentration of 32 hours) were recommended for use instead of a synthetic unit hydrograph with a storm duration of a time of concentration of 42 hours;

Figure 2. Aerial view of Hardap Dam during the 2006 flood event.

Table 1. Representative flood peaks (Hattingh: 2007).

Probability of exceedance (%)	Flood peak (m^3/s) Original design	Recommended
50.0%		466
20.0%		1 267
10.0%	1 690	1 908
5.0%		2 623
2.0%	2 840	3 720
1.0%	3 120	4 689
0.5%		5 803
0.2%	5 240	7 537
0.1%	6 400	9 080
SEF		11 640

- The 2006 flood event had a flood peak of 3 967 m^3/s, a flood volume of 458 million m^3 and a probability of exceedance of 1.64%; and
- Other extreme events occurred in 1972 (6 826 m^3/s), 1974 (2 996 m^3/s) and 2000 (2 914 m^3/s).

2.3 *Sensitivity analysis*

A sensitivity analysis was done to highlight the importance of a sufficiently long record when using the results of statistical analysis especially for the design of a dam.

The sensitivity analysis involved the determination of flood peaks using different record lengths (10, 20, 30, 40 and 50 years respectively) over the complete 65 year record. For example for a record length of 50 years one can determine 16 different flood peak distributions, for a record length of 40 years 26 different flood peak distributions etc.

It is important to note that the flood peaks for each distribution were calculated using the results of a statistical analysis using the General Extreme Value (GEV) distribution and that only the original record was used. The results therefore are a reflection of the characteristics of the actual record. A next step in this sensitivity analysis is to perform a stochastic analysis on the original record and then to perform a sensitivity analysis on the results of the stochastic analysis. This is not discussed in this paper.

The results of the sensitivity analysis (the distributions of the relative correlation with the full 65 year record for flood peaks with probabilities of exceedance of 1%, 0.1% and 0.01%) are provided in Figures 3 to 5. From the results for this particular site with this particular 65 year record the following is evident:

- A record of more than 30 years is needed for comparable flood peaks starting in any given year over the 65 year record;
- Record lengths of 30 years and less could result in estimated flood peaks varying from 60% lower to 70% higher than flood peaks determined from the full record;
- For the original 18 year record the flood peak with probability of exceedance of 1% was 38%

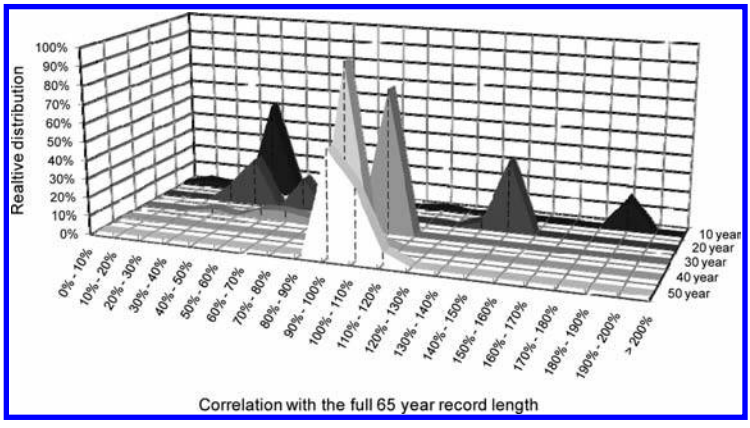

Figure 3. Results of the sensitivity analysis for flood peaks with a probability of exceedance of 1%.

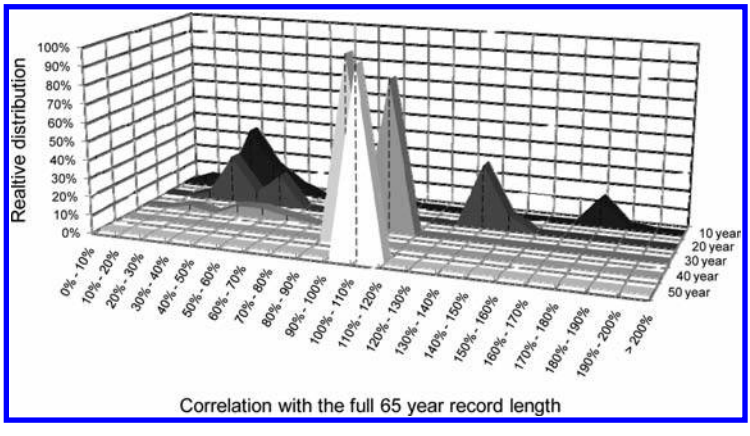

Figure 4. Results of the sensitivity analysis for flood peaks with a probability of exceedance of 0.1%.

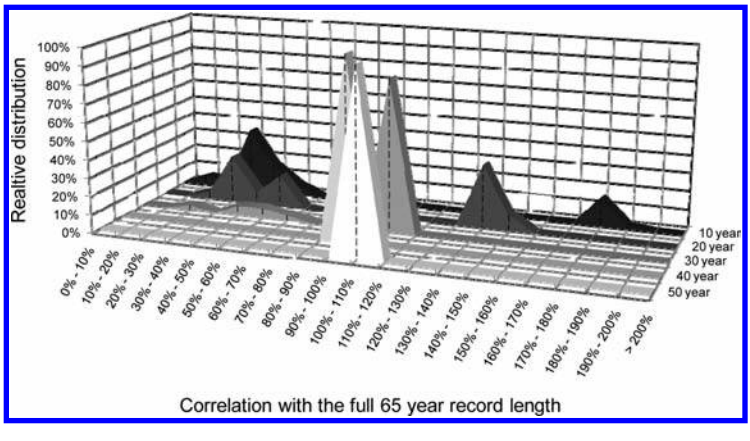

Figure 5. Results of the sensitivity analysis for flood peaks with a probability of exceedance of 0.01%.

less than for the full record and the flood peak with probability of exceedance of 0.01% was 20% less; and
- There is no distinct difference between the results of flood peaks with relative high probabilities of exceedance (1%) and those with relative low probabilities of exceedance (0.01%).

2.4 *Dam safety assessment*

From the results of the dam safety assessment (Hattingh 2007b) the following was evident:

- The spillway capacity of the dam does not satisfy current day criteria as the flood routing analysis (using level pool routing) indicated that the spillway system with the radial gates functioning, cannot accommodate the routed SEF without overtopping of the non-overspill crest. It can only accommodate a flood peak equal to 8 500 m^3/s (probability of exceedance of about 0.15%) without overtopping of the non-overspill crest of the main embankments. It is important to note that the secondary embankment as well as the saddle structures would be overtopped;
- In addition, the flood routing analysis also indicated that the spillway system with all the radial gates closed, can only accommodate a flood peak equal to 3 800 m^3/s (probability of exceedance of about 2%) without overtopping of the non-overspill crest; and
- The spillway gates increase the risk posed by the dam (see Figures 6 & 7 respectively) based on the methodology used by the Department of Water Affairs of South Africa (Oosthuizen et al. 2002, Oosthuizen & Hattingh 2006).

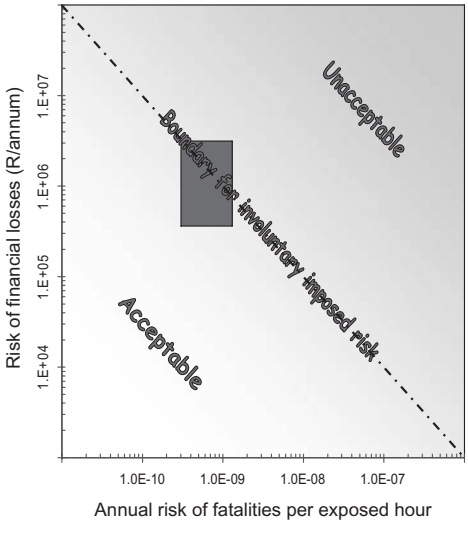

Figure 6. Risk of Hardap dam as if it has been built without flood gates.

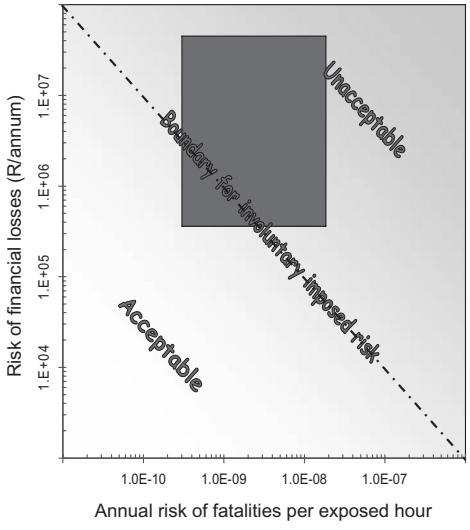

Figure 7. Risk of the existing Hardap dam (with flood gates).

3 FLOOD SITUATION ASSESSMENT AND SOLUTIONS

3.1 *Background*

The town of Mariental is 17 km south south east of Hardap Dam. The irrigation scheme is situated between the dam and Mariental.

The capacity of the existing downstream river channel (overgrown with reeds) has been calibrated on site by actual releases as 500 m^3/s prior to the 2006 flood event while the channel capacity without reeds has been estimated to be between 1 400 and 1 600 m^3/s using hydrodynamic modelling (Dunn 2007).

Public perceptions and media reports following the flood event created the impression that the damage to the irrigation scheme and Mariental town was caused by the incorrect operation of Hardap Dam.

An independent assessment of the operation of Hardap Dam (Hattingh 2007c) concluded that the flood routing decisions followed the prescribed methodology, all possible actions were taken to ensure the least possible damage to Mariental Town and the irrigation area during the 2005/2006 flood season and that the releases (outflows) from Hardap Dam were always less than the inflows. This was done notwithstanding difficult conditions

during the flood event including a power failure and difficulties with the backup generator during the flood peak.

3.2 *Solutions*

A number of possible solutions were identified to mitigate future flood damage to Mariental and the irrigation scheme during the assessment (Hattingh 2007d). They were:

- The "do nothing" scenario with the government of Namibia taking responsibility for all future losses;
- Moving Mariental town out of the flood plain;
- Clearing the overgrown river channel downstream of the dam without damaging the riverine vegetation;
- Flood control reservoirs upstream of Hardap Dam as well as in the tributary catchments downstream of the dam to attenuate the 1% flood peaks to the different river channel capacities;
- Raising of Hardap Dam to increase its attenuation capacity to attenuate the 1% flood peaks to the different river channel capacities and satisfy dam safety requirements; or
- A combination of these options.

The "do nothing" solution was no longer considered feasible as this was the chosen solution after the previous extreme flood events of the 1970's and 2000. As a result the insurance industry of Namibia made a decision to no longer insure any losses in Mariental or the irrigation scheme.

The residents of Mariental were not in favour of the option of moving the town out of the flood plain due to the negative social and environmental impacts associated with such a move.

The option to increase the attenuation of Hardap Dam included the removal of the existing radial gates on the central spillway to reduce the risk (see Figures 6 & 7), raising the fixed ogee concrete sill, closing off some of these central spillway sections (either none, one, two, three or all four of the sections), replacing the secondary earthfill embankment as well as the auxiliary spillway with a concrete ogee or labyrinth spillway and raising all the embankments to the required non-overspill crest levels for each option using the existing slopes.

From the results of a prefeasibility analysis, that included the options of the flood control reservoirs and increasing the attenuation of Hardap Dam, the following was evident:

- Only the option of constructing a 50 m high flood control reservoir upstream of Hardap Dam is sufficient, together with Hardap Dam, to attenuate the SEF without overtopping of Hardap Dam;
- The unit cost per attenuation unit is basically the same for all the options of raising of Hardap Dam whereas for the flood control dams it decreases as the reservoir size increases;
- When comparing the results from the cost calculations for both the flood control reservoir upstream of Hardap Dam and the raising of Hardap Dam it is clear that the raising of Hardap Dam would be the more cost effective (see Figure 8). It is also important to note that

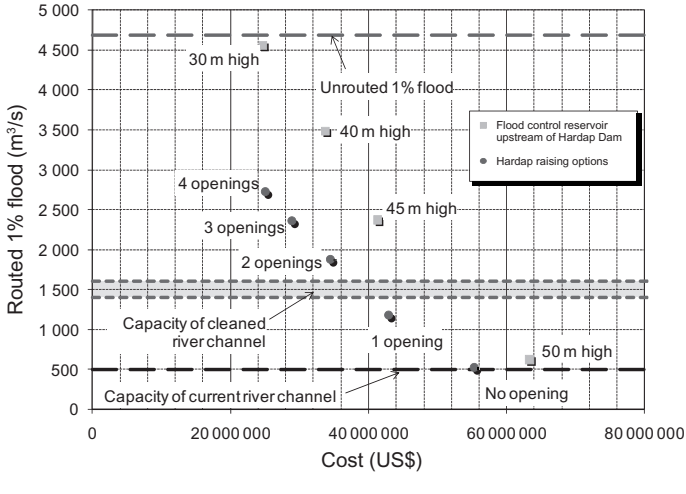

Figure 8. Comparison between flood control reservoir upstream of Hardap Dam and the different raising options of Hardap Dam.

the raising option also includes the upgrading of the spillway capacity to accommodate the SEF peak;
- Closing three of the four openings and raising of Hardap Dam will attenuate the 1% flood peak sufficiently for the downstream river channel cleared of all reeds to accommodate the flood;
- Closing all four spillway openings and raising of Hardap Dam is sufficient to attenuate the 1% flood peaks sufficiently for the existing downstream river channel to accommodate the routed flood without overtopping of its banks; and
- It is estimated that it will cost US$ 6 million to only improve the spillway capacity of Hardap Dam to handle the extreme flood events and therefore satisfy only the dam safety requirements.

4 CONCLUDING REMARKS

The use of an 18 year hydrological record in the design of the dam resulted in the spillway capacity being under designed and therefore not satisfying current day dam safety criteria. The results of the sensitivity analysis of the current hydrological record highlights the significance of long enough records as a record of more than 30 years is needed for comparable flood peaks starting in any given year over the 65 year record for the whole range of flood peak probabilities of exceedance.

For this specific record there also is no distinct difference between the results of flood peaks with relative high probabilities of exceedance (1%) and those with relative low probabilities of exceedance (0.01%).

The combined solution of cleaning the river channel downstream of Hardap Dam and improving the attenuation capacity of Hardap Dam (through the removal of the radial gates, closing three of the four spillway openings, raising the existing ogee spillway, the addition of a concrete ogee spillway on the left flank replacing the secondary earthfill embankment and the auxiliary spillway and raising all the embankments at least 10.4 m) was the most cost effective solution to protect Mariental town without moving it out of the flood plain and the downstream irrigation scheme as well as improve the spillway capacity to satisfy current day dam safety criteria. The remedial work to Hardap Dam is managed through MAWF.

ACKNOWLEDGEMENTS

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Studying and forecast of the Nurek reservoirs sedimentation taking into account of lateral inflows

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ABSTRACT: The prognoses and actual values of sedimentation of the Nurek reservoir analyzed and present analytical expressions for determine of the reservoirs volume by use of the date hydrological date of the hydrological stations are presented. Results of calculation lateral inflow to the Nurek reservoir are presented.

1 ANALYTICAL CALCULATIONS OF THE NUREK RESERVOIRS SEDIMENTATION

For analytical calculations were used daily values of key parameters of Nurek HydroPower Station (HPS): the discharge of water passed through the turbines of HPS (Q_1, m^3/sec), single outlet (Q_0, m^3/sec), water inflow to the reservoir (Q_3, m^3/sec), mark of the upstream reach (H_0, m). For this purpose by chosen any base point (W_0; H_0) other points of a curve can be received directly on the equations:

$$W(H_{i+1}) = W(H_i) + W_a \cdot f \quad (1)$$

$$W_a = [Q_3 - (Q_1 + Q_0 + Q_2)] \cdot t \quad (2)$$

where, t = 1 month = n · 24 · 3600 sec., n—month's days.

As base for all further calculations we accept point $W_0 = 10.5$ km^3 (volume of the reservoir) and $H_0 = 910$ m corresponding to a design condition of the reservoirs. It geometrically means parallel carrying over of actual curves of volume of a water basin to one general beginning and allows comparing them visually. It is clear that such calculations can be made only in a range of fluctuations of level of a reservoir really taking place in operation that is such analysis is possible only for useful volume of a reservoir.

For calculation of evaporation and the reservoir area following dependences has been used accordingly:

$$Q_2(t) = k \cdot S \quad (3)$$

$$S = 0.39408 \cdot H - 267.86306 \quad (4)$$

where, S—area of a reservoir, km^2

Values of the coefficient (k) are accepted on the direct supervision spent on a reservoir during enough long periods. By use of (3) and (4) were calculated dependences of the water level from volume of a reservoir for 2003 which is shown on Fig. 1. Similar calculations were spent as well for 2004 which are almost identical to the Fig. 1. These calculations were spent both for daily and for the monthly average hydrological data.

The comparative analysis of all available curves of the dependence of the water level (H) from volume (W) of the Nurek reservoir is shown on Fig. 2 which demonstrated that curves of H = f(W) calculated according to operational services of HPS for 2003–2004 confirm results of bathymetric measurements of 2001 years.

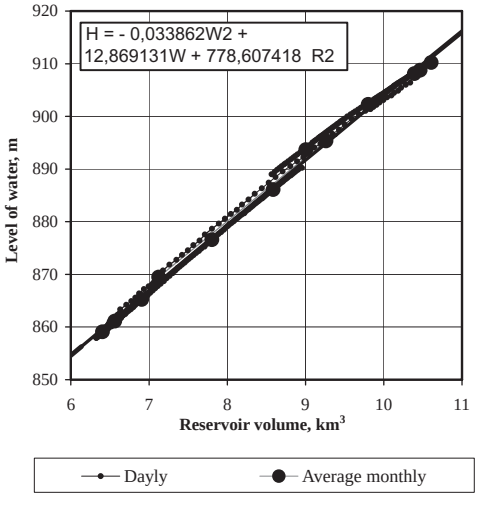

Figure 1. Calculation of the H = f(W) on the data of the Nurek HPS for 2003 year.

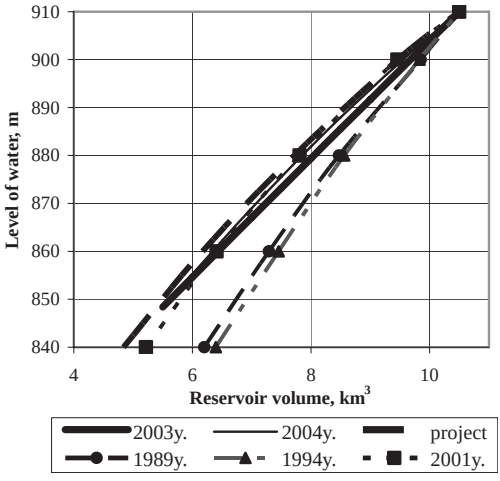

Figure 2. The comparative analysis of the dependence H = f(W).

2 CALCULATION OF LATERAL INFLOW TO THE NUREK RESERVOIR ON THE SITE FROM THE KOMSOMOLABAD HYDROPOST TO NUREK HPS

The Komsomolabad hydropost on which measurements of the Vakhsh river discharge are made is located on distance of 70 km above on a current from the Nurek reservoir. Lateral inflow on the considered site (a hydropost of Komsomolabad hydropost—Nurek HPS) can reach several tens cubic meter per second or more 5% from the total inflow. It is necessary to notice that direct measurements of the lateral inflows after 80th years of the last century were not spent and not spent now (Normatov, Froebrich etc., 2006).

For definition of value lateral inflows of the next years we will take advantage of the data presented in Table 1.

The presence of very close connection between mid-annual values of the lateral inflow to the Nurek reservoir and mid-annual water discharge on the hydropost (Fig. 3) was revealed.

On the other hand absolute values of lateral inflow have square-law dependence with mid-annual values of the water discharge and expressed in percentage—linear. By considering above mentioned accuracy of the close connection for the further calculations following expression was accepted:

$$Q_6 = f(Q_7) = 0{,}00011Q^2_7 - 0.01693Q_7 - 3.49861 \quad (5)$$

where Q_6, Q_7—mid-annual values of the water discharge of lateral inflows and Vakhsh rivers on the Komsomolabad hydropost, m^3/sec, accordingly.

Absence of connection between monthly values of the lateral inflow discharge and the water discharge of the Vakhsh River on the Komsomolabad hydropost at simultaneous presence of good connection between their mid-annual discharges explain by shift of their hydrographs in time (Normatov & Petrov, 2005). It is well seems on Fig. 5 which shows that spring floods and low water on the Vakhsh river in comparison with lateral inflows are late for two months. It explains by the big difference of heights of territories on which that and other drain is formed.

More exact annual distribution of lateral inflow discharge (relations of monthly average values to mid-annual) for 1985–1987 is resulted in Table 2.

As appears from Table 2 approximation of the hydrograph lateral inflow hydrograph can be executed with enough accuracy by use of the equations:

$$q_0/Q_6 = \varphi(n) = 0.01995479n^4 - 0.37785543n^3 + 2.28314161n^2 - 4.64468079n + 3.27093041 \quad \text{for } n = 1 - 8, \quad (6)$$

$$q_0/Q_6 = \varphi(n) = 0.00970911n + 0.39090805 \quad \text{for } n = 9 - 12 \quad (7)$$

where, n—number of month in year, q_0—monthly average discharge of lateral inflows, m^3/sec.

Taking into account these dependences the definitive formula for calculation of monthly average discharge of the lateral inflows to the Nurek reservoir can be written as:

$$q_0 = f(Q_7) \cdot \varphi(n) \quad (8)$$

In (8) the first factor depends only from the mid-annual water discharge of the Vakhsh River, Q_7) and the second—only from a season (number of month, n). The calculated values φ(n) are resulted in the Table 3.

The calculated and actual values of the lateral inflows for 1985–1987 years are summarized in Table 4 which show good convergence of calculated and actual values of lateral inflow. It is necessary to notice that average values calculated and actual of lateral inflows differ on 0.08 m^3/sec. Thus their average difference absolute value is equal, 4.01 m^3/sec. It less than one percent (0.66%) from the mean annual water discharges of the Vakhsh River.

Thus the developed method of calculation of the lateral inflow to the Nurek reservoir has quite sufficient accuracy for technical calculations. But thus it is necessary to notice that the method assumes that is already known the mid-annual водность of the Vakhsh rivers—water discharge Q_7. Therefore such calculations can be

Table 1. Actual values of lateral inflows to the Nurek reservoir in 1985–1987 years.

Years	1985		1986		1987	
m	A, m^3/sec	B, m^3/sec	A, m^3/sec	B, m^3/sec	A, m^3/sec	B, m^3/sec
I	193.40	12.10	165.80	12.00	167.60	13.70
II	170.70	12.60	163.70	12.20	165.30	14.20
III	207.60	34.40	163.90	14.40	234.10	51.90
IV	463.00	69.40	274.70	26.20	459.10	85.60
V	825.10	85.50	491.70	37.10	888.20	91.90
VI	1207.6	44.40	802.10	23.20	1570.60	74.80
VII	1736.0	23.10	1374.00	11.00	1577.40	57.50
VIII	1306.8	12.10	1116.00	9.20	1561.00	18.90
IX	628.90	10.50	646.20	7.60	869.20	16.20
X	309.50	13.30	328.20	7.70	455.90	20.00
XI	212.20	12.70	230.30	10.20	292.00	16.20
XII	192.60	12.40	190.00	11.60	229.00	13.10
Aver.	621.12	28.54	495.55	15.20	705.78	39.50
%	–	4.59	–	3.07	–	5.60

A- Discharge on hydropost; B- Lateral inflow, m- months

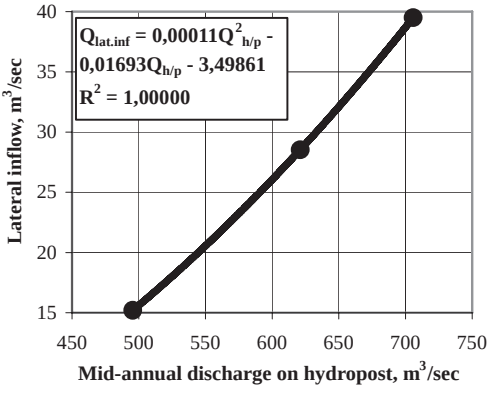

Figure 3. Dependence of lateral inflow from the mid-annual water discharge on the hydropost.

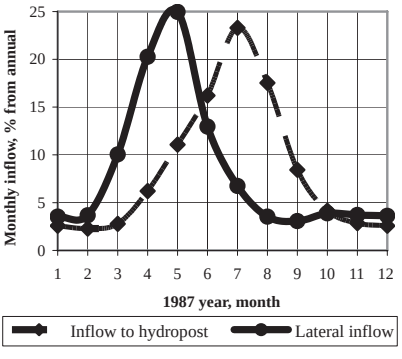

Figure 4. Hydrographs of the Vakhsh River on komsomolabad hydropost and lateral inflow on the side Komsomolabad hydropost—Nurek HPS.

Table 2. Hydrograph of lateral inflow ($Q_{mon.aver.}/Q_{mid-ann}$).

	Month											
Year	I	II	III	IV	V	VI	VII	VIII	IX	X	XI	XII
1985	0.42	0.44	1.20	2.43	2.10	1.56	0.81	0.42	0.37	0.47	0.45	0.43
1986	0.79	0.80	0.95	1.72	2.44	1.53	0.72	0.61	0.50	0.51	0.67	0.76
1987	0.35	0.36	1.31	2.17	2.33	1.89	1.46	0.48	0.41	0.51	0.41	0.33

Table 3. Calculated value of φ(n).

n	I	II	III	IV	V	VI
φ(n)	0.55	0.41	1.30	2.15	2.37	1.84
n	VII	VIII	IX	X	XI	XII
φ(n)	0.94	0.51	0.48	0.49	0.50	0.51

Table 4. Comparison of calculated and actual values of lateral inflow.

y	1985			1986			1987		
	Q, m³/sec			Q, m³/sec			Q, m³/sec		
m	Q_{ac}	Q_{cal}	Q_{ac}/Q_{cal}	Q_{ac}	Q_{cal}	Q_{ac}/Q_{cal}	Q_{ac}	Q_{cal}	Q_{ac}/Q_{cal}
1	12.10	15.70	0.77	12.00	8.40	1.43	13.70	21.81	0.63
2	12.60	11.69	1.08	12.20	6.25	1.95	14.20	16.24	0.87
3	34.40	37.00	0.93	14.40	19.78	0.73	51.90	51.39	1.01
4	69.40	61.16	1.13	26.20	32.70	0.80	85.60	84.96	1.01
5	85.50	67.36	1.27	37.10	36.02	1.03	91.90	93.57	0.98
6	44.40	52.41	0.85	23.20	28.02	0.83	74.80	72.79	1.03
7	23.10	26.74	0.86	11.00	14.30	0.77	57.50	37.14	1.55
8	12.10	14.45	0.84	9.20	7.72	1.19	18.90	20.07	0.94
9	10.50	13.62	0.77	7.60	7.28	1.04	16.20	18.92	0.86
10	13.30	13.89	0.96	7.70	7.43	1.04	20.00	19.30	1.04
11	12.70	14.17	0.90	10.20	7.58	1.35	16.20	19.68	0.82
12	12.40	14.45	0.86	11.60	7.72	1.50	13.10	20.07	0.65

Note: Q_{ac}, Q_{cal}—actual and calculated values of lateral inflows m- months.

spent or for the retrospective analysis in particular for bathygraphic calculations or for planning of the future work of the HPS, but at reliable enough forecast of the rivers hydrological parameters.

3 CALCULATIONS OF VOLUME OF THE NUREK RESERVOIR ON THE HYDROLOGICAL DATA OF THE KOMSOMOLABAD HYDROPOST

Bathiografic calculations of the Nurek reservoirs corrected on lateral inflows for 2003 years are executed in the present section. It is possible by using (Normatov etc., 2006) value of the mid-annual water discharge of the Vakhsh river on an alignment of the Komsomolabad hydropost (Q_7 = 613.09 m³/sec) to receive mid-annual value lateral inflows on the part from the Komsomolabad hydropost to Nurek HPS:

$$Q_6 = f(Q_7) = 0.00011Q^2_7 - 0.01693Q_7 - 3.49861 = 27.47$$

Using values φ(n) from Table 4 it is possible to receive monthly average values of the lateral inflows q_0 necessary for the further calculations:

m	I	II	III	IV	V	VI
q_0, m³/s	15.1	11.3	35.7	59.0	65.0	50.6
m	VII	VIII	IX	X	XI	XII
q_0, m³/s	25.8	13.9	13.1	13.4	13.7	13.9

The calculated curve of dependence H = f(W) on Fig. 6 and comparison of the given curve

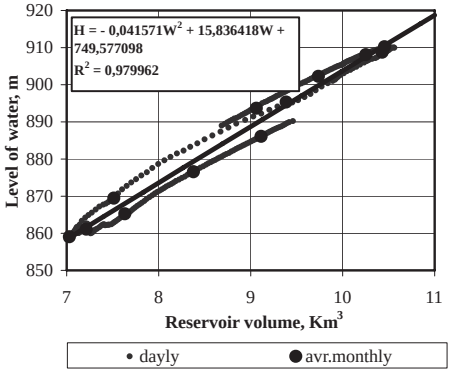

Figure 5. Calculation value of H = f(W) on the Nurek HPS data of 2003 year.

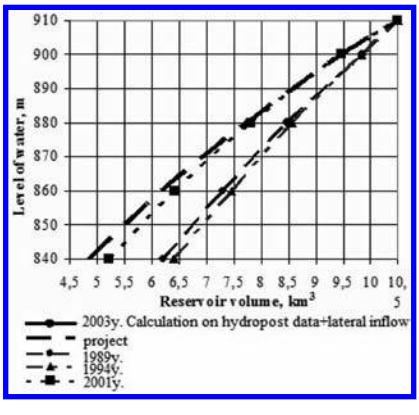

Figure 6. The comparative analysis of H = f(W) dependences.

with others received with above mentioned dependences on Fig. 5 is presented.

It is clear from Fig. 7 that at use of inflows data from Komsomolabad hydropost corrected on lat lateral inflows the calculated curve of volumes shows essential sedimentation of the useful volume of the Nurek reservoir. The calculated curve for 2003 years in this case practically repeats a curve received at direct bathymetric measurements of 1989 years.

Hence the curve on Fig. 6 now is the most reliable for all calculations connected with a reservoir and the method of calculation of the bathygraphic.

Characteristics developed in the present work can be used for monitoring of sedimentation process of the useful volume—the most important for Hydropower station operation.

CONCLUSION

The analytical method of the curves of the reservoir volume calculation has enough high accuracy and allows to operatively changes of morphometric characteristics of the reservoir within its useful volume. But for their practical application it is necessary to provide with accuracy of all entering into the equation of water balance of the initial parameters: inflow, outlet, losses.

For increase of the Nurek HPS efficiency and sedimentation process control of the reservoir are necessary the reliable hydrological data on lateral inflows. It is possible to provide it at use of the supervision materials of the Komsomolabad hydropost and also at restoration of the hydropost in Rogun HPS.

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Medidas ambientales para preservar la calidad del embalse de La Colada

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RESUMEN: El embalse de La Colada tiene la función de abastecimiento de agua a poblaciones, por lo que tiene que cumplir unos rangos mínimos de calidad sanitaria. Actualmente nos encontramos con la problemática de la existencia de instalaciones de ganadería en la zona del valle de los Pedroches, que contaminan los suelos y cuyas sustancias acaban finalmente en la red hidrográfica que drena hacia el embalse. Por otro lado, la capacidad de tratamiento de las estaciones depuradoras de las poblaciones que drenan al embalse es insuficiente para los caudales que aportan las industrias, los municipios y los aprovechamientos ganaderos. Con objeto de evitar la entrada de aguas de deficiente calidad en el embalse se va a desarrollar una actuación en uno de los arroyos principales, el Guadarramilla, que consiste en el bombeo de los caudales que provienen de esta subcuenca para evitar que entren en el embalse. Con esto se consigue la monitorización y regulación de los caudales de bombeo en el arroyo Guadarramilla para preservar la calidad del agua almacenada.

ABSTRACT: The Colada reservoir has the function of water supply to populations, it has to fulfill a few minimal ranges of sanitary quality. Nowadays we meet the problematics of the existence of facilities of ranching in the zone of the valley of Pedroches, it contaminate the soils and whose substances end finally in the hydrographic network that drein towards the reservoir. On the other hand, the capacity of treatment of the stations filter systems of the populations that drein to the reservoir is insufficient for the flows that contribute the industries, the municipalities and the cattle utilizations. In order to avoid the water entry of deficient quality in the reservoir one is going to develop an action in one of the principal creeks, the Guadarramilla, which consists of the pumping of the flows that come from this subbasin to prevent them from entering the reservoir. With this there is obtained the monitoring and regulation of the flows of pumping in the creek Guadarramilla to preserve the quality of the stored water.

1 INTRODUCCIÓN

El embalse de La Colada tiene como finalidad principal la de abastecimiento a los municipios de la Comarca de los Pedroches, situada en la zona norte de Córdoba y los de la Comarca de Almadén (Ciudad Real), que en conjunto representan en estos momentos una población de 85.000 habitantes. Actualmente nos encontramos con la problemática de la existencia de instalaciones de ganadería en la zona del valle de los Pedroches, que contaminan los suelos y cuyas sustancias acaban finalmente en la red hidrográfica que drena hacia el embalse. Por otro lado, la capacidad de tratamiento de la EDAR de Pozoblanco es insuficiente para los caudales que aportan las industrias, el municipio y los aprovechamientos ganaderos de la zona. También la EDAR de El Viso y Villaralto está en pruebas, sin un funcionamiento efectivo. El embalse de La Colada tiene la función de abastecimiento de agua a poblaciones, por lo que tiene que cumplir unos rangos mínimos de calidad sanitaria.

Con objeto de evitar la entrada de aguas de deficiente calidad en el embalse se va a desarrollar una actuación en uno de los arroyos principales,

Figura 1. Presa de La Colada.

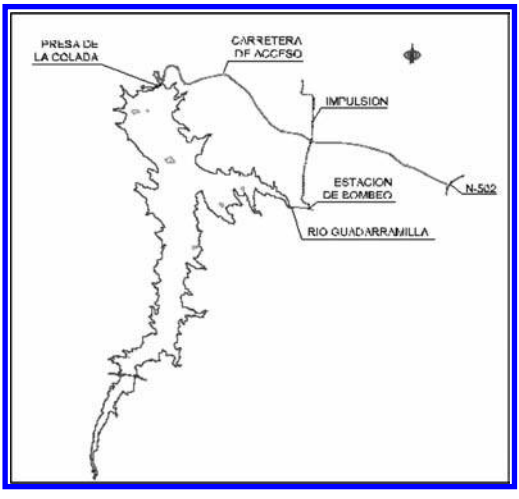

Figura 2. Situación de las obras.

el Guadarramilla, que consiste en el bombeo de los caudales que provienen de esta subcuenca para evitar que entren en el embalse. Con esto se consigue la monitorización y regulación de los caudales de bombeo en el arroyo Guadarramilla para preservar la calidad del agua almacenada. Se contempla el bombeo de aguas pluviales sobre el arroyo Guadarramilla, que es uno de los dos aportes principales que recibe la presa de La Colada, junto con el Guadamatilla, sobre el que se sitúa la citada presa. De este modo, y hasta que la calidad del agua que afluye al embalse por este arroyo esté garantizada en su integridad (especialmente los caudales de estiaje) podrán by-pasearse éstos, evitando su incorporación al embalse y el consiguiente empeoramiento en la calidad del agua del embalse.

La plataforma y edificio de bombeo se encuentran sobre una de las colas del embalse, por encima de la cota de máxima inundación, donde hay construido un pequeño azud de hormigón que permitirá remansar el agua y captar lateralmente mediante un bombeo los caudales ordinarios y mediante una tubería de impulsión salvar la divisoria para derivarlos aguas abajo de la presa mediante su incorporación al arroyo Cerro Blanco, evitando así la incorporación de caudales con elevada carga de contaminación.

2 ACTUACIONES PREVISTAS

La finalidad principal del Embalse de La Colada es el abastecimiento de agua a los municipios de la Comarca de los Pedroches, en Córdoba, desempeñando además otras funciones como el abastecimiento a la Comarca de Almadén, en Ciudad Real, la regulación del río Guadamatilla, la laminación de avenidas, así como el mantenimiento del régimen de caudales ecológicos. La presa se ha construido recientemente sobre el río Guadamatilla, con 48.5 m de altura con una capacidad de embalse de 57.7 Hm^3 que recibe una aportación media anual de 32 Hm^3 de los que pueden regularse para abastecimiento 13.96 Hm^3 con una garantía del 99.72%. El embalse con una superficie de 530 ha (NMN) ocupa terrenos en los municipios de Belalcázar, el Viso, Villaralto e Hinojosa del Duque, en la provincia de Córdoba. La obra fue licitada, en diciembre de 2001, iniciándose su construcción el 16 de mayo de 2002. En la actualidad las obras están concluidas y se están llevando la fase de explotación y llenado del embalse en la forma y condiciones establecidas en el Reglamento Técnico sobre Seguridad de Presas y Embalses de 1996.

Debido a las numerosas granjas de ganadería (vacuno y porcino) existentes en la zona y a la incapacidad para tratar toda al agua contaminada de la EDAR de Pozoblanco (con una capacidad máxima de 200 m^3/h), los caudales con gran aporte orgánico se concentran en el Guadarramilla.

Figura 3. Azud del Guadamatilla.

Figura 4. Eutrofización del embalse.

3 LOS CAUCES

El cauce objeto de este estudio, el Guadarramilla, recibe, entre otros, los residuos generados por las numerosas explotaciones ganaderas que se ubican en las proximidades del río y sus afluentes, constituyendo un grave peligro de contaminación de las aguas y degradación del entorno.

En las zonas de dominio público, servidumbre y policía de los cauces vertientes al embalse se han detectado 58 explotaciones ganaderas y se han censado 6.510 cabezas de ganado vacuno. La mayor parte de las instalaciones ganaderas carecen de sistemas apropiados para la gestión de residuos, por lo que en la mayoría de los casos, se incorporan directamente al dominio público hidráulico sin ningún tipo de tratamiento depurador, con el consiguiente deterioro.

Por otro lado, la EDAR de Pozoblanco es insuficiente, siendo capaz de depurar únicamente la mitad del caudal que accede a la depuradora. El resto se vierte al Guadarramilla sin depuración alguna.

Estas importantes cantidades de nitrógeno y fósforo contribuirán a la eutrofización del embalse, con un crecimiento elevado de algas y otros microorganismos, que son descompuestos por la actividad bacteriana, lo que supone un gasto importante de oxígeno. Las aguas resultan turbias, con malos olores, resulta difícil la vida piscícola y se reduce drásticamente su calidad para el consumo humano.

Figura 5. Explotación ganadera junto al embalse.

Tabla 1. Clasificación de aguas del CEDEX.

CLASIFICACIÓN DE AGUAS DEL CEDEX

NIVELES DE CALIDAD

PARAMETROS	UNIDAD	I	II	III	IV
O_2 disuelto	% sobre saturación	> 80	60 - 80	40 - 60	< 40
DBO_5	mg/l O_2	< 3	3-5	5-7	> 7
Materia en suspensión	mg/l MES	< 25	< 25	< 25	> 25
Amoniaco	mg/l NH_4^+	< 0,5	0,5 - 1	1 - 2	> 2
Nitratos	mg/l NO_3	< 50	< 50	< 50	> 50
Fosfatos	mg/l P_2O_5	< 0,5	0,5 - 1	0,5 - 1	> 1
Clorofila	mg/m^3	< 10	10 - 20	20 - 50	> 50
Índice biótico	Índice GMWP	> 100	60 - 100	35 - 60	< 35
Cloruros	mg/l Cl	< 200	< 200	200 - 500	> 500
Conductividad	µS/cm a 20 °C	< 750	750 - 1500	1500 - 3000	> 3000
Hierro	mg/l Fe	< 0,5	0,5 - 1	1 - 2	> 2
Sustancias extraíbles al cloroformo	mg/l SEC	< 0,1	0,1 - 0,5	0,5 - 1,0	> 1,0
Coliformes totales	100 ml a 37 °C	< 5000	5000 - 10000		> 10000
Coliformes fecales	100 ml	< 2000	< 2000		> 2000
Estreptococos fecales	100 ml	< 1000	1000 – 10000		> 10000
Color	mg Pt/l	< 20	20 - 100	100 - 200	> 200
pH		El pH deberá estar comprendido entre 6 y 9.			
Temperatura		No deberá exceder de 25 °C. Se tendrán en cuenta las especiales condiciones para ríos salmonícolas.			
Espumas y residuos flotantes		No perceptibles.			

4 CALIDAD DE LAS AGUAS

En el Anejo I del R.D. 927/88 de 29 de julio, por el que se aprueba el Reglamento de la Administración pública del Agua y de la Planificación Hidrológica titulado "Calidad exigida a las aguas superficiales destinadas a la producción de agua potable", se expone textualmente:

I. Las aguas superficiales susceptibles de ser destinadas al consumo humano quedan clasificadas en los tres grupos siguientes, según el grado de tratamiento que deben recibir para su potabilización:
   - Tipo A_1: Tratamiento físico simple y desinfección.
   - Tipo A_2: Tratamiento físico normal, tratamiento químico y desinfección.
   - Tipo A_3: Tratamiento físico y químico intensivos, afino y desinfección.

II. Los niveles de calidad de las aguas superficiales destinadas a la producción de aguas potable que fijen los Planes Hidrológicos, no podrán ser menos estrictos que los que figuran en la tabla siguiente para los distintos tipos de calidad figurados en el apartado anterior, salvo que se prevea un tratamiento especial que las haga potables.

En virtud de lo establecido en el artículo 48 del Plan Hidrológico I de la Cuenca del Guadiana, la calidad mínima exigible a las aguas, en un tramo de un río o acuífero con tomas para abastecimiento urbano, será del tipo A_2, fijado en el Anexo I (R.D. 927/1988).

El propio Plan, define los objetivos de calidad por zonas y tramos de río, correspondiendo al río Guadamatilla la Zona 4, subzona 4.13.1.

Según el contenido del Plan las aguas del río Zújar aguas presentan una calidad excelente que cumple con holgura con los requerimientos mínimos de calidad marcados para el abastecimiento y el riego. Por ello, se requiere tan sólo del control de las prácticas agrícolas y de los vertidos industriales de la zona. El objetivo de calidad es el mantenimiento de los niveles actuales que varían entre el nivel I y el II según la clasificación del CEDEX, dado que sus usos principales son el abastecimiento y el riego. En concreto, en la zona donde se encuentra el embalse de La Colada, el nivel de calidad objetivo según la clasificación del CEDEX es nivel II.

Puesto que el destino de las aguas del embalse de La Colada es precisamente el de abastecimiento, se exige que la calidad de las mismas se corresponda con el tipo A_2.

4.1 *Valoración del agua y cursos fluviales*

Dentro del inventario y su valoración se incluye el análisis de la calidad del agua, partiendo de los

Tabla 2. Clasificación de aguas.

PARAMETRO	UD	A_1	A_2	A_3
PH		6,5–8,5	5,5–9	5,5–9
Temperatura	ºC	25	25	25
Conductividad a 20ºC	Us/cm	1000	1000	1000
Nitratos	mg/l	50	50	50
Cloruros	mg/l	200	200	200
Fosfatos	mg/l	0,4	0,7	0,7
DBO_5	mg/l	3	5	7
O_2 disuelto	mg/l	70	50	30
Amoniaco	mg/l	0,05	1,5	4
Manganeso	mg/l	0,05	0,1	1
Cobre	mg/l	0,05	0,05	1
Zinc	mg/l	3	5	5
Sulfatos	mg/l	250	250	250
Coliformes totales	100 ml	50	5000	50000
Coliformes fecales	100 ml	20	2000	20000
Fluoruros	mg/l	1,5	1,7	1,7
Boro	mg/l	1	1	1
Arsénico	mg/l	0,05	0,05	0,1
Cadmio	mg/l	0,005	0,005	0,005
Cromo total	mg/l	0,05	0,05	0,05
Plomo	mg/l	0,05	0,05	0,05
Selenio	mg/l	0,01	0,01	0,01
Bario	mg/l	0,1	1	1
Cianuros	mg/l	0,05	0,05	0,05
Hidrocarburos disueltos	mg/l	0,05	0,2	1

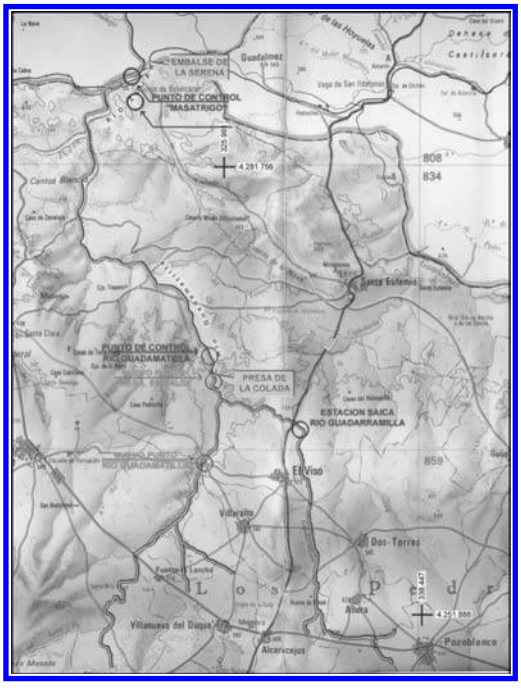

Figura 6. Situación de los puntos de control.

datos analíticos recogidos en el "Control Intensivo de la Calidad del Agua Embalsada destinada al Abastecimiento de la Población en la Cuenca Hidrográfica del Guadiana", elaborado por la propia Confederación Hidrográfica del Guadiana en verano de 1993.

En dicho estudio se recogen datos en la cola del embalse de La Serena (a 40 km aguas abajo de la presa de La Colada) en el llamado viaducto de "Masatrigo", y en el río Guadamatilla a unos 2 km de la presa. Estos datos son completados en el Estudio de Impacto Ambiental de la presa con dos controles de agua en la desembocadura del río Guadamatilla y en el río Guadarramilla, en las fechas 28/03/1995 y 06/04/95. En la figura adjunta se muestra la situación de estos puntos.

4.2 *Análisis físico químico*

En la Serena las aguas están considerablemente mineralizadas, con conductividades que llegan a los 500 micromhos/cm. Estos valores alcanzan los 1.650 micromhos/cm en la desembocadura del Guadarramilla, y 1.540 micromhos/cm en la del Guadamatilla, excediendo los límites marcados por el R.D. El pH es claramente alcalino, alcanzando el valor de 8,9 en La Serena, pero disparándose a 9,5 en la desembocadura del Guadamatilla.

4.3 *Análisis químico*

Mientras que en La Serena los valores de cloruros, sulfatos, nitratos, calcio, hierro y demás características químicas analizadas se encuentran dentro de las especificaciones más rigurosas, no sucede así en los otros dos cauces. Tanto en el Guadarramilla como en el Guadamatilla se detectaron contenidos de cloruros muy altos (hasta 278,3 mg/l Cl-). Los contenidos de manganeso obligan a un tratamiento químico (0.75 mg/l en el Guadarramilla). El boro es también elevado y el arsénico supera los límites reglamentarios en ambos ríos (0,158 mg/l en el Guadamatilla y 0,195 mg/l en el Guadamatilla).

4.4 *Análisis químico-biológico*

En La Serena, el fósforo se mantiene en concentraciones discretas, oscilando entre los 0,01 y 0,07 mg/l. Sin embargo, tanto en el Guadamatilla como especialmente en el Guadarramilla, el contenido de fósforo es muy alto superando los 0,7 mg/l marcados por la ley (2,60 y 40,40 mg PO43-/l respectivamente).

En cuanto al carbono, la tendencia que se observa en La Serena es un aumento de las concentraciones presentes a medida que avanza el periodo estival, al final del cual se alcanza un valor de Carbono Orgánico Total (COT) de 31,5 mg/l. El valor

Figura 7. Verano de 2008, eutrofización del embalse.

Figura 8. Recogida de lenteja de agua en el embalse, en el verano de 2008.

mínimo registrado es de 7,58 mg/l y la media alcanza los 19,6 mg/l. Estas cifras no son excesivamente altas salvo al final del periodo, lo que induce a esperar una moderada productividad primaria.

La evolución de la fracción purgable es paralela al COT, y las concentraciones detectadas son sensiblemente superiores a las de la fracción no purgable, lo que indica la presencia en el medio de moléculas orgánicas no complejas, y por consiguientes, fácilmente asimilables por el sistema.

La tendencia del nitrógeno en las muestras analizadas en La Serena es de ir en aumento a medida que avanza el verano. Los valores extremos registrados son de 0,25 y 3,79 mg/l de Nitrógeno Total.

Con respecto a la clorofila los valores registrados son especialmente elevados, predominando la clorofila A sobre la B y C, lo que indica, por una parte una actividad fotosintética activa motivada por la presencia de nutrientes en el medio, y por otra, una determinada preponderancia de determinadas comunidades fitoplactónicas presentes. Los valores encontrados indican condiciones de eutrofia-hipertrofia.

4.5 *Análisis de plaguicidas*

Prácticamente hay ausencia de plaguicidas halogenados, nitrogenados y fosforados tanto en La Serena como en los ríos Guadamatilla y Guadarramilla.

4.6 *Oxígeno disuelto*

Se mantienen concentraciones elevadas de oxígeno disuelto en el embalse de La Serena y en el río Guadamatilla. Sin embargo, en el río Guadarramilla los valores detectados (35,2 y 39,6) son especialmente bajos y próximos al límite inferior marcado por el R.D.

La DQO 10 min alcanza en la Serena al final del mes de agosto el valor de 14,0 mg/l O_2. La DBO alcanza el valor de 8,2 mg/l de O_2 en el Guadamatilla y de 11,0 mg/l de O_2 en el Guadarramilla, ambos excesivamente elevados para permitir la vida piscícola.

4.7 *Conclusiones del Estudio de Impacto Ambiental*

Las aguas que abastecerían en el futuro el embalse de La Colada mostraban ya en época del Estudio de Impacto Ambiental un estado de hipertrofia. En el propio estudio, achancan la baja calidad de las aguas a los vertidos de la localidad de Pozoblanco, de 15.000 habitantes, que en ese momento, carecía de estación depuradora de aguas residuales.

La analítica realizada a las muestras tomadas en los tres puntos (La Serena, desembocadura del Guadamatilla y desembocadura del Guadarramilla) indican que las aguas eran ya de tipo A_3 de los indicados en el Anejo 1 del R.D. 927/88, es decir, que necesitan para su potabilización un tratamiento físico y químico intensivos, afino y desinfección.

A la vista de los resultados obtenidos en los análisis de calidad de aguas de los ríos Guadamatilla y Guadarramilla, el principal problema que se plantea en las aguas del embalse es el de la eutrofización en la masa de agua del futuro embalse.

Con objeto de evitarlo, se proponían las siguientes actuaciones:

– Construcción de un dique en la cola del embalse en el río Guadarramilla. El objeto del mismo es retener y sedimentar la mayor parte de la materia orgánica que contienen sus aguas, al objeto de no incorporar dicha materia orgánica al embalse.

Durante la construcción del mismo no deben eliminarse la vegetación de ribera de esta zona y se recomienda la realización de una plantación de álamos y olmos. La revegetación constituirá un sistema de depuración natural de los caudales circulantes por el río Guadarramilla y retendrá las fracciones más gruesas de los sedimentos. Por otro lado, los acarreos retenidos aterrarán el dique sirviendo de esta forma de sustrato de aguas poco profundas que retendrán parte de los contaminantes orgánicos, potenciando la vegetación de ribera.

– Deforestación integral del área inundada, ejecutándose esta en aquellos meses que no interfieran o se solapen con el ciclo biológico de las comunidades animales.

El plan de Vigilancia Ambiental incluye un programa de seguimiento de los parámetros de contaminación de la masa de agua de embalse.

El propio Plan establece unos valores límite necesario para asegurar la existencia de fauna ictícola aguas abajo de la presa, que se detallan a continuación:

– Temperatura máxima 13ºC
– pH 7–8,5
– Conductividad máxima 500 microS/cm
– Materia en suspensión máxima 750 mg/l
– Oxígeno disuelto mínimo 5 mg/l
– DBO máximo 3 mg/l
– Cloruros 250 mg/l
– Sulfatos máximos 250 mg/l
– Nitratos máximos 50 mg/l
– Fosfatos máximos 5 mg/l P_2O_5
– Plomo máximo 0,2 mg/l
– Zinc máximo 3 mg/l
– Hidrocarburos máximo 0,2 mg/l.

Por otro lado y para controlar el nivel de eutrofia del embalse propone no exceder los siguientes valores:

– Concentración de clorofila A < 8 mg/m^3
– Profundidad de Secchi >2 m.

Por último, se propone que, durante los primeros años de la fase de explotación del embalse, se lleven a cabo unos análisis periódicos mensuales (principalmente durante los meses de verano) para el control de la eutrofización potencial por descomposición de los restos vegetales no deforestados o inmersos dentro de la masa de agua.

5 ESTUDIO DE INUNDABILIDAD EN EXPLOTACIONES GANADERAS

Con motivo de la elevada concentración de explotaciones ganaderas en las cuencas de los ríos Guadamatilla, y Guadarramilla, la Confederación Hidrográfica del Guadiana solicitó a los titulares de cada una de ellas un estudio con objeto de definir las áreas inundables por las avenidas correspondientes a distintos periodos de retorno.

Se detectaron 58 explotaciones ganaderas con un censo de 6.510 cabezas de ganado vacuno. El 71% de las explotaciones se sitúan en el río Guadarramilla y sus afluentes, con una producción anual de 72.553 toneladas de estiércol, lo que supone una producción de 290.212 kg de nitrógeno y 145.106 kg de P_2O_5.

La mayor parte de las instalaciones ganaderas carecen de sistemas apropiados para la gestión de residuos, por lo que en la mayoría de los casos, se incorporan directamente al dominio público hidráulico sin ningún tipo de tratamiento depurador, con el consiguiente deterioro. Además, los residuos almacenados en las instalaciones pueden ser arrastrados en caso de presentación de alguna de las avenidas.

Con estos trabajos se pretendía determinar los niveles de avenida alcanzados por cada periodo de retorno para definir las zonas de dominio público, servidumbre y policía, con el objeto final de permitir las obras necesarias para acondicionar cada una de las instalaciones minimizando los vertidos y mejorando la calidad de las aguas.

Teniendo en cuenta que, en la cuenca de recepción del embalse de La Colada, destaca por su aportación, el río Guadamatilla (curso principal) y el río Guadarramilla, que tributa al anterior, la Confederación Hidrográfica del Guadiana ha realizado análisis de calidad de agua en tres puntos:

– Embalse de La Colada, junto a la presa
– Río Guadamatilla (cruce con la carretera de El Viso a Hinojosa del Duque)
– Río Guadarramilla (cruce con la carretera de El Viso a Santa Eufemia y próximo a la estación de la red SAICA).

Para determinar la aptitud de las aguas para el baño y el uso deportivo se consideran las concentraciones que esta agua presenta en determinados parámetros. En este caso se presta especial atención a los recuentos microbiológicos, el porcentaje de saturación de oxígeno, y en menor medida, presencia de aceites y grasas y otros caracteres organolépticos (olor, sabor, etc). Para determinar la aptitud de las aguas para la vida piscícola influye mucho la concentración de nitritos y también el amoniaco no ionizado, que es muy tóxico para los organismos acuáticos, aún a bajas concentraciones; y también, aunque menos, la DBO5, amonio, hidrocarburos disueltos y metales (Pb, Cu, Zn) presentes.

Figura 9. Aportaciones extraordinarias en diciembre de 2009.

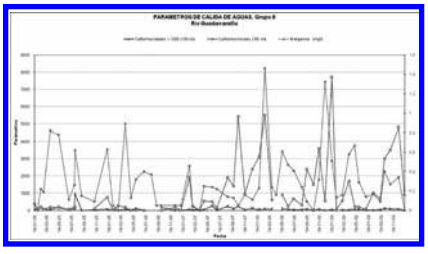

Figura 10. Evolución de los parámetros de calidad del agua.

5.1 *Análisis físico—químico*

Las aguas del río Guadamatilla se consideran mineralizadas, con conductividades que llegan a alcanzar los 1156 micromhos/cm; valores que aumentan hasta los 1476 micromhos/cm en el río Guadarramilla, sobrepasando los límites legales permitidos. En los análisis efectuados en la presa, se han superado los 1000 micromhos/cm desde septiembre de 2004 a marzo de 2006, con una máxima de 1242 micromhos/cm.

El pH es claramente alcalino, alcanzando el valor 8,9 en el Guadamatilla, y 8,7 en la desembocadura del Guadarramilla. En la presa el valor máximo detectado asciende a 9,8, siendo superado el límite del tipo A_3, en el mes de junio.

Los valores encontrados de cloruros, sulfatos, nitratos, calcio, hierro y demás características químicas analizadas se encuentran dentro de las especificaciones más rigurosas. Sólo se ha detectado un incremento de los cloruros durante los meses de verano de 2005 en el río Guadarramilla.

Sin embargo, en ambos ríos se han encontrado contenidos en fosfatos muy altos, con lo que supera los niveles de calidad marcados, de 0,7 mg/l. El contenido en manganeso es también significativo en el caso del río Guadarramilla (máx 0,89 mg/l), lo que obligaría a un tratamiento de descontaminación químico.

En cuanto al arsénico, el contenido en la presa, es superior al límite permitido, siendo superior a 0,07 mg/l desde mayo de 2005 a enero de 2006.

5.2 *Análisis químico—biológico*

No se han detectado plaguicidas halogenados, nitrogenados y fosforados. Los detectados aparecen de forma puntual y concentraciones traza.

5.3 *Oxígeno disuelto*

Mientras que los valores de oxigeno disuelto se encuentran dentro de la normalidad en los tres puntos, no sucede lo mismo con la DBO5 que alcanza valores de hasta 19,9 en la presa y 47,2 en el río Guadarramilla, estando el valor medio superando el valor de 5 indicado en la legislación para calidad de aguas del tipo A_2.

5.4 *Conclusiones*

En esta campaña de muestreo la analítica realizada demuestra que las aguas están más próximas al tipo A_3, es decir, que necesitan previamente a su consumo un proceso de depuración físico y químico intensivo así como desinfección.

Según los parámetros considerados en la Directiva del Consejo 78/659/CEE, de 18 de julio, relativa a la calidad de las aguas continentales que requieren protección o mejora para ser aptas para la vida de los peces, los niveles registrados de algunos parámetros superan los umbrales señalados en el Anexo I de la citada Directiva.

En concreto, el valor marcado por los parámetros pH, DBO5 y Fósforo total, supera los niveles de cumplimiento obligatorio o recomendado por esta.

Puesto que el origen de la contaminación es principalmente orgánico y el foco contaminante está bien situado, parecer factible mejorar a corto plazo la calidad de las aguas de este curso si se toman las adecuadas medidas.

Asimismo se ha considerado la importancia de la contaminación difusa adicional, difícilmente controlable, procedente de las granjas situadas a lo largo del río Guadarramilla.

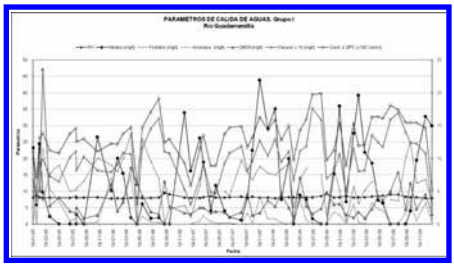

Figura 11. Evolución de los parámetros de calidad del agua.

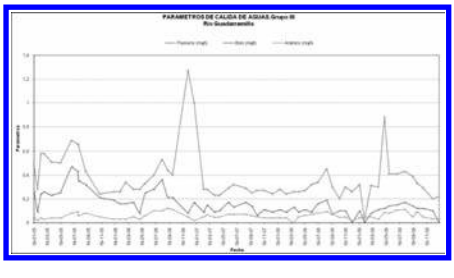

Figura 12. Evolución de los parámetros de calidad del agua.

6 DIQUE DE DEPURACIÓN NATURAL

Como ya se ha indicado, una de las medidas correctoras de protección de calidad de las aguas incluidas en el EIA del proyecto de la presa, era la creación de un dique de cola de embalse en el río Guadarramilla.

El objeto de este dique es la retención y sedimentación de la mayor parte de la materia orgánica que contienen sus aguas, al objeto de no incorporar dicha materia orgánica al embalse, constituyendo un sistema de depuración natural de los caudales circulantes por el río Guadarramilla y reteniendo las fracciones más gruesas de los sedimentos.

6.1 *Proyecto inicial*

El dique se componía, según proyecto, de una cimentación de hormigón en masa, sobre la cual se construyen tres líneas de obra de anchura cada vez menor y, a medida que se ensancha el barranco, dimensión longitudinal mayor. La obra culmina con un vertedero de labio fijo que centra el rebosamiento del agua sobre el cimiento, para evitar erosiones en la zona de empotramiento.

La obra se proyecta de mampostería de piedra del lugar con objeto de integrarla en el entorno.

La altura del dique en su centro es de 2,5 m siendo su longitud en coronación de 60 m.

Se recomienda además no eliminar la vegetación de ribera de la zona y complementarla con una plantación de álamos y olmos, desde 50 m aguas abajo del dique hasta 500 maguas arriba del mismo.

6.2 *Obras realizadas*

Las obras del azud en el río Guadarramilla finalizaron en julio de 2005.

Se cambio la ubicación aprovechando un salto natural del río. La altura no supera 1,50 m. La longitud de vertido es de 9,15 m y conforma un azud de hormigón en masa que se sitúa sobre el cauce elevando la cota a la 477,53. El resto de la coronación hasta una longitud de 27,28 m, se completa con un dique de tierras hasta la cota 478,50.

Figura 13. Dique en río Guadarramilla. Planta.

Ambas partes son separadas por medio de una plataforma de hormigón, también a cota 478,50, que sirve a su vez de protección contra erosiones a la salida del azud.

A la vista de los ensayos de calidad de aguas realizados tras las obras en el río Guadarramilla se concluye que las obras realizadas no son suficientes para mejorar la calidad de las aguas.

La mayor parte de los parámetros analizados o se mantiene constantes o incluso aumentan en valor, es decir, los efectos esperados no se ha producidos (ver gráficos del apartado 4.2). Cabe destacar que en la actualidad se ha procedido a la restauración ambiental y el plantado de álamos y olmos tal y como especificaba la DIA, por lo que la efectividad esperada del dique no puede confirmarse aún.

Sin embargo si se prevé que la contaminación global de las aguas en la presa no mejore todo lo esperado. Por ello se propone la realización de las siguientes medidas complementarias:

- Bombeo de caudales contaminados retenidos en el dique al río Guadamatilla, a un punto situado aguas abajo de la presa.
- Creación de una zona de lagunaje junto al dique para depuración natural de la aguas por evaporación.

Esta última opción se ha descartado por la elevada superficie de lagunaje necesaria, los inevitables malos olores, y el sobrecoste que supone la gestión de los resides finales.

La solución del bombeo de caudales aguas abajo de la presa parece, en un principio, trasladar el problema de la contaminación aguas abajo de la presa. Sin embargo si se analizan los resultados obtenidos en los análisis de calidad de agua efectuados para el EIA del proyecto, se observa que la mala calidad de agua registrada en el río Guadarramilla y Guadamatilla se ve minimizada en la cola del embalse de La Serena, siendo importante el poder de autodepuración del cauce y del propio embalse de La Serena.

Cabe mencionar, que esta solución es concomitante con la mejora de la calidad de agua de los

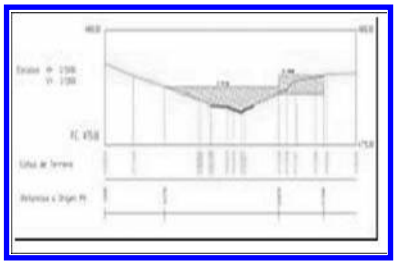

Figura 14. Dique en río Guadarramilla. Perfil longitudinal.

vertidos a los largo del cauce del río Guadarramilla, causados tanto por la EDAR de Pozoblanco como por las explotaciones ganaderas que a él vierten.

7 ESTACIÓN DE BOMBEO

Al estar situado el embalse de La Colada en la confluencia de los ríos Guadamatilla y Guadarramilla, y presentar cada uno de ellos unas características diferentes en cuanto a la calidad de sus aguas, a solicitud de Hidroguadiana, la Comisaría de Aguas ha realizado campañas de toma de muestras y analíticas en tres puntos diferentes: en ambos cauces antes de la confluencia así como en el propio embalse creado por la ataguía del desvío del río.

Como consecuencia de los resultados obtenidos se detecta una mayor concentración de contaminantes en el arroyo Guadarramilla, cuya cuenca vertiente es aproximadamente el 20% del total de la cuenca del embalse que la constituye principalmente el río Guadamatilla. Esto se debe principalmente a que la cuenca del río Guadarramilla recibe el vertido de la depuradora de Pozoblanco, que entró en funcionamiento en el año 2000, pero que en la actualidad presenta problemas de capacidad de tratamiento. Por otra parte, también existen un número significativo de explotaciones ganaderas próximas al cauce del Guadarramilla, que han sido objeto de un programa de actuaciones para evitar el arrastre de contaminación que ocasiona la escorrentía por dichas explotaciones. Además, se está finalizando la EDAR de El Viso -Villaralto que actualmente vierten directamente al Guadarramilla y Guadamatilla respectivamente, pero que una vez puesta en funcionamiento, el vertido depurado lo hará al río Guadamatilla y en caso de fallo se derivará el vertido al Guadarramilla.

Por tanto, el riesgo de vertido no depurado al embalse está concentrado en el río Guadarramilla, y es por lo que se ha proyectado una solución que permite, por las condiciones topográficas e hidrográficas existentes, el derivar los caudales contaminados que se detecten en el Guadarramilla hacia el arroyo Cerro Blanco, afluente del arroyo Linarejo, que desemboca en el río Guadamatilla 1.900 metros aguas abajo de la presa de La Colada. Y ello es viable, aprovechando la retención que produce el dique de cola del Guadarramilla antes de incorporarse al embalse, donde se instalará una estación de bombeo de 280 l/s, con capacidad de evacuar un caudal 4 veces superior al caudal medio de aguas residuales generado en la cuenca vertiente, evaluado en 70 l/s, salvando así la divisoria de la cuenca del Guadarramilla con tan solo una altura manométrica de 60 m.c.a. y una longitud de impulsión de 1.020 m y un tramo en gravedad de 1.732 m, con una acometida eléctrica desde la línea aérea ya instalada de suministro a la presa de tan solo 50 metros de longitud.

El bombeo se diseñado con un equipamiento 4 grupos motobombas verticales para aguas residuales de 90 kW, con tres etapas, lo que permite elevar un caudal unitario de 70 l/s a 60 m.c.a, salvando la divisoria intermedia y rompiendo carga en arqueta de rotura.

La plataforma de bombeo se proyecta como un losa de hormigón armado de 40 cm de espesor a cota 481, no inundable para la avenida de 100 años, descansando sobre seis fustes circulares de 50 cm de

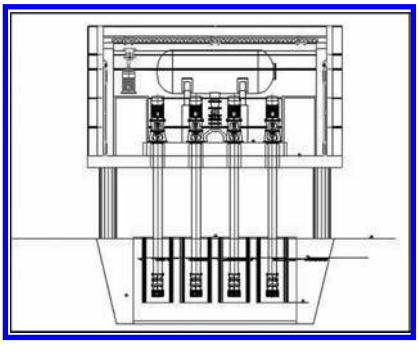

Figura 16. Estación de bombeo en río Guadarramilla. Alzado.

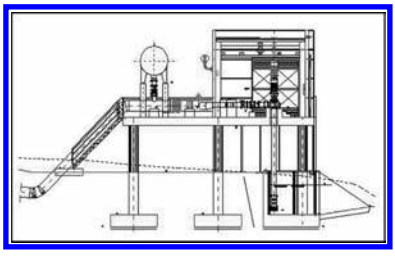

Figura 15. Estación de bombeo en río Guadarramilla. Perfil.

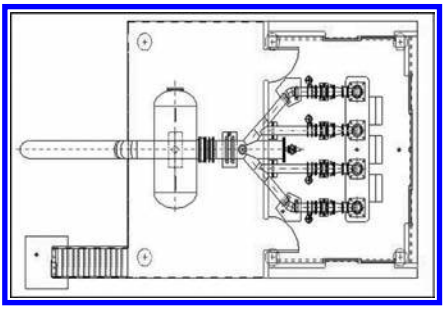

Figura 17. Estación de bombeo en río Guadarramilla. Planta.

diámetro cimentados directamente sobre zapatas individuales de 2,5 × 2,5 m que se apoyan sobre un fondo rocoso. A la plataforma se accede mediante una escalera metálica para lo que se proyectado un camino y una plataforma de acceso.

Las bombas se disponen de forma simétrica, 2 a 2, sobre la plataforma anterior. Las impulsiones individuales van provistas de todo el equipamiento necesario para su correcto funcionamiento: juntas de compensación, válvulas antiretorno, carretes de desmontaje, válvulas de mariposa motorizas y anclajes y apoyos donde se requieren.

El colector general va provisto igualmente de junta de compensación que la aisla del resto de la conducción y un carrete de desmontaje. Se ha previsto igualmente un calderín hidroneumático de vejiga de 5 m^3 que consigue amortiguar correctamente el transitorio generado ante una parada brusca por corte en el suministro eléctrico.

A escasos 20 metros de la plataforma de bombeo se ha habilitado una explanación para situar el centro de transformación y el centro de control, ambos de tipo prefabricado. La acometida eléctrica se realiza desde la línea aérea existente para suministro eléctrico a la presa de La Colada que discurre por las proximidades y consta de un tramo aéreo de 51 m y un tramo subterráneo de unos 40 m hasta alcanzar el centro de transformación.

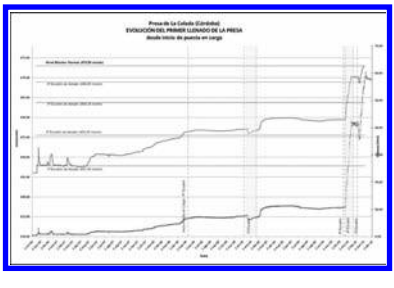

Figura 18. Evolución del llenado de embalse con una simulación de bombeo de by-pass.

Figura 19. Tipha Latifolia en las inmediaciones del Azud del Guadarramilla.

En esta zona se sitúa también una arqueta caudalímetro para control del volumen bombeado. La regulación del bombeo se podrá realizar a partir de los niveles en el azud.

8 CONCLUSIONES

Se ha realizado una simulación de un posible bombeo con los caudales de aportación obtenidos de la serie completa de la que se dispone desde octubre de 2006 hasta marzo de 2010, con los datos reales de los caudales aforados en el Azud del Guadarramilla. En el caso más desfavorable en el cual habría que bombear todos los volúmenes circulantes, el proceso de llenado del embalse se hubiera llevado a término sin ningún problema, ya que aunque representa un porcentaje apreciable, se prevé su utilización con caudales de estiaje.

Se ha realizado un balance de caudales aportados al embalse de la Colada, ponderando y estimando con la relación de superficies, los caudales que han entrado por el Guadarramilla al embalse, teniendo en cuenta curva de llenado, volúmenes, salidas y evaporación. Se ha estimado cuando funcionaría y cuando no el bombeo en base a los caudales aportados.

Por otro lado, se han relacionado estos caudales con las tablas y gráficos de los datos de calidad y completamos con los datos más actuales. Existe una buena correlación en la mejora de la calidad con el incremento de caudal aportado.

Aunque la serie no es muy larga, son datos reales, y es cierto que cuando se producen entradas por encima de los 300 l/s aproximadamente se aprecia ostensiblemente la mejora en los parámetros de calidad observados.

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Estaciones de aforo: Base del sistema de información hidrológica y de la gestión del recurso hídrico

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ABSTRACT: Gauge stations, as a part of a hydrologic network, are the essential tools to obtain an accurate knowledge of the water flowing through rivers and channels. The choice of a suitable location, typology, instrumentation and maintenance of gauge stations allows, together with a meticulous and systematic calibration, to get precise real data about the discharge evolution in chosen control sections.

In particular, good quality hydrometric data are the basis not only to design dams, making appropriate its reservoir capacity and outlet devices to each catchment basin, but also to optimise exploitation through an effective water resource management, including flood situations, provided by an Automatic Hydrologic Information System (SAIH) and a Decision Support System (DSS). This operational tool in the Ebro river basin collects meteorological forecasts and observed data (rainfall, temperature, river flows from the gauge stations and reservoir water levels and outflows) and use hydrological and hydraulic models, besides reservoir operation, to obtain flood forecasts.

El conocimiento de los caudales circulantes por los ríos y canales y de las reservas en los embalses de una cuenca hidrográfica, constituye el elemento básico para la gestión del recurso hídrico.

La hidrometría proporciona los datos de nivel y caudal requeridos para la elaboración de las series estadísticas que, entre otras aplicaciones, permiten realizar una adecuada planificación hidrológica, estudios de inundabilidad o dimensionamiento de obras hidráulicas e infraestructuras relacionadas con los cauces.

El registro y medición de los niveles de agua en los cauces y sus caudales correspondientes se lleva a cabo mediante unas instalaciones fijas denominadas estaciones de aforo.

Las estaciones de aforo son secciones de control situadas en un tramo adecuado de un cauce, natural o artificial, en las que se obtiene el dato del nivel (expresado en metros) de la lámina de agua circulante referido a una escala limnimétrica fija y su caudal asociado (expresado en m^3/s) obtenido generalmente mediante una curva gastos elaborada para esa sección.

En la cuenca del Ebro existen actualmente más de 300 estaciones de aforo, situadas en ríos, canales y embalses, explotadas por el Servicio de Aforos del Área de Hidrología y Cauces de la Confederación Hidrográfica. Dichas estaciones se integran en la Red Oficial de Estaciones de Aforo (09-ROEA) que el Ministerio de Medio Ambiente y Medio Rural y Marino, a través de su Dirección General del Agua, gestiona en todas las cuenca hidrográficas españolas mediante las Confederaciones Hidrográficas, y que proporcionan los datos de caudal y reservas publicados en los Anuarios de Aforos. Existen además otras estaciones de aforo propiedad de Comunidades Autónomas, empresas hidroeléctricas y otras entidades.

Adicionalmente, la práctica totalidad de las estaciones de aforo de la cuenca del Ebro (09-ROEA) se han incorporado al Sistema Automático de Información Hidrológica. La transmisión de los datos foronómicos en tiempo real a los centros de proceso de cuenca, posibilita el seguimiento y control de los niveles, caudales y reservas tanto en situaciones normales de explotación como en avenidas.

La elección del emplazamiento de una estación de aforos viene determinada por la necesidad de conocer el caudal circulante real en determinados tramos de ríos como, desembocaduras, confluencias, entradas y salidas de embalses, poblaciones significativas, etc. Localmente, la ubicación concreta de la sección de medida está condicionada por las características físicas del tramo: pendiente, anchura, profundidad, accesos, afecciones a la corriente, etc. De este modo, las estaciones de aforo presentan diferentes tipologías, desde las situadas en cauce libre hasta las que disponen de canales con vertedero, adecuadas a las distintas secciones de medida.

En todo caso, todas las estaciones de aforo que se proyectan y diseñan en la cuenca del Ebro son

de propósito general. Es decir, el objetivo de su instalación es obtener datos continuos y precisos de toda la gama de niveles y caudales circulantes por la sección de medida, tanto en épocas de estiaje como en situaciones de avenida.

Los datos de caudal en una estación de aforo se calculan generalmente a partir del registro continuo del nivel alcanzado por el agua, referido a una escala limnimétrica fija, realizado mediante limnígrafos o sensores de diversa tipología conectados a recolectores de datos. Este nivel se transforma en caudal a través de una tabla de tarado denominada curva de gastos.

La curva de gastos debe ser una ley de transformación biunívoca, altura-caudal, válida para una sección de características determinadas y durante un período de tiempo en el que se mantengan inalteradas las condiciones de fluencia del agua en el tramo de aforo. Es decir, el ajuste de las curvas de gasto en las estaciones de aforo es un proceso continuo que requiere una revisión sistemática a través de un recalibrado mediante los aforos directos de comprobación. En cada una de las estaciones de aforo de la cuenca del Ebro estos aforos directos se realizan una vez cada tres meses y siempre que se produzcan situaciones de cambio en las secciones de medida tales como avenidas, obras, etc.

En los casos en los que la relación altura-caudal no es biunívoca, por ejemplo casos de estaciones en ríos influenciadas por mareas, en canales afectados por compuertas, etc., además del nivel es preciso acudir a la medición de otra variable, generalmente la velocidad en un punto, en una banda o la velocidad media de la sección de desagüe, con la instalación de los sistemas precisos, y con este dato y el de nivel poder determinar el caudal de manera fidedigna.

El cálculo de la curva de gasto para una estación de aforo, que permite obtener los caudales circulantes a partir de los niveles registrados, se realiza a partir de fórmulas hidráulicas (Manning) o modelos matemáticos (HEC-RAS) en los que se introducen datos de la sección mojada, perímetro mojado, coeficiente de desagüe, rugosidad del cauce, pendiente motriz, etc. Los coeficientes de ajuste se estiman a partir de los datos de los aforos directos más altos. Las curvas así obtenidas se ajustan finalmente a los aforos directos correspondientes a su época de vigencia por procedimientos manuales.

Es fundamental por tanto que el diseño de los tramos de aforo y de todas las instalaciones de una estación de aforos contenga los detalles hidrométricos necesarios para que las condiciones de circulación del agua se parezcan lo más posible a las condiciones teóricas ideales requeridas por las fórmulas que se utilizan en el cálculo de los tarados nivel-caudal o curvas de gasto.

Este diseño, debe estar basado no sólo en los requerimientos teóricos del sistema de cálculo, sino en la experiencia hidrométrica práctica, con lo que se consigue un funcionamiento óptimo de la estación de aforo, lo que permite obtener datos de nivel y caudal de mejor calidad. Una recomendación muy común es la de que la estación se instale en un tramo recto, que implica generalmente la existencia de la profundidad máxima en el centro. Este diseño teóricamente bueno no lo es desde el punto de vista práctico. Es más adecuado que el tramo presente una ligera curvatura, con lo que la profundidad máxima se encuentra próxima a la margen cóncava, lo que proporciona innegables ventajas a la hora de realizar la instalación de la escala limnimétrica y el pozo de registro para instalar el sistema boya-sirga-contrapeso, todavía hoy el más seguro para registros de datos hidrométricos de nivel.

Los detalles de diseño de cada elemento de la instalación: soleras, muros de encauzamiento y de acompañamiento, vertederos, pozo, pasarela y andador para aforos directos, accesos, etc. son numerosos y fundamentales para asegurar la fiabilidad y continuidad homogénea en las series de datos registradas, así como para facilitar las labores del personal que debe vigilar, comprobar el funcionamiento, realizar aforos directos y labores de mantenimiento y puesta a punto en las estaciones, además de no influir de manera negativa en el ecosistema, especialmente en el paso de la ictiofauna. En el caso de simplificaciones o meras copias incompletas del diseño de una estación de aforo pueden producirse desviaciones importantes en las estimaciones de caudal.

Un ejemplo lo tenemos en la historia de la estación de aforos, número 101, sobre el río Aragón aguas abajo del embalse de Yesa. La antigua estación, Figura 1, presentaba discrepancias erráticas con los datos de las sueltas medias y altas del embalse. El motivo de estas diferencias radicaba en el diseño, ya que el pozo de registro de niveles

Figura 1. E.A. nº 101, río Aragón en Yesa Pie de Presa. Antigua instalación.

se encontraba situado en una pequeña pila de puente unida a la margen mediante una pasarela. El efecto pila hacía que en determinadas condiciones de fluencia el incremento de nivel sobre el de la circulación general, producido por la presencia de la pila, afectaba a los niveles registrados en el pozo; en otros momentos, resultaba afectado por la depresión que el mismo fenómeno produce, lo que se traducía en unos caudales distorsionados que unas veces resultaban más grandes que los reales y otras menores. En la actualidad la caseta con el pozo de registro se ha trasladado a la margen derecha, unificándose con la del teleférico y la del SAIH, con lo que la medida del nivel no resulta alterada al encontrarse el pozo de registro tras el muro marginal que conduce el agua sin interferencias, los vertederos han sido remodelados y el teleférico para aforos directos se ha reformado aumentando su seguridad, Figura 2.

Una vez obtenida la serie estadística de los caudales de cada estación de aforo, ha de ser contrastada con datos de otras estaciones próximas (balances), especialmente en episodios de avenidas, y con las precipitaciones registradas en la cuenca de aportación (previa restitución al régimen natural). Por ejemplo, si en una estación de aforos situada próxima a la desembocadura de un afluente a un río principal se registra una avenida importante, de difícil aforo directo, y al desembocar al cauce principal alcanza valores altos en la estación de aforos próxima de aguas abajo bien medidos con aforos directos, el dato calculado en el cauce principal permite ajustar la parte alta de la curva de gastos en el tributario con mucha precisión.

Las series históricas de datos foronómicos no presentan siempre la misma calidad ni homogeneidad. Los diferentes criterios y posibilidades económicas y tecnológicas a lo largo de los casi 100 años de producción de datos para los anuarios de aforo, hacen necesario contrastar y corregir las series históricas cuando sea posible, mediante diversos procedimientos, como los hidrológicos-estadísticos u otros, con el fin de validarlas antes de su utilización. Un ejemplo: durante ciertos años la curva de gastos era única durante el año hidráulico y se realizaba con todos los aforos directos del año, que podían pertenecer a dos ramas distintas si durante ese año se producía un cambio en el tramo, por depósito de sedimentos o por arrastre de los mismos. El resultado es que los datos quedan afectados durante un tiempo por exceso y durante otro por defecto, distorsionándose los mínimos, los máximos, y en el mejor de los casos el caudal medio anual se podía parecer al real. Actualmente el cambio de rama o curva de gastos se realiza en el momento que se detecta la desviación, procurando elegir el día más acorde con las circunstancias que han producido la variación en la relación altura-caudal, como puede ser el día siguiente al registro del máximo nivel de avenida.

Figura 2. E.A. nº 101, río Aragón en Yesa Pie de Presa. Nueva instalación.

Los errores más frecuentes que se detectan en las series estadísticas de niveles y caudales se producen en los valores extremos: máximos y mínimos. En muchas ocasiones estos datos erróneos se deben a un deficiente o inadecuado diseño de las estaciones de aforo. Los datos de estiaje pueden ser distorsionados fácilmente por el nacimiento de algas en las soleras de los vertederos o por el depósito de piedras por personas que en verano pretenden darse un baño. Los datos de avenidas pueden tener errores producidos por la definición de las curvas de gasto. Por ejemplo: durante una época la parte alta de las curvas se extrapolaba linealmente a partir del tramo alto dibujado sobre los aforos directos, ignorando una causa de modificación tan simple y fácil de obtener como es la sección topográfica de desagüe que pasa por la escala hidrométrica, ya que los aforos directos con los que se traza la curva de gastos se realizaban siempre con el río dentro del cauce y nunca en situación de desbordamiento, lo que producía una extrapolación con errores apreciables.

En otras ocasiones un diseño inadecuado, como puede ser la existencia de múltiples muros divisorios de la sección de aforo con la finalidad de tener varios tramos de vertederos con diferentes profundidades y velocidades del agua fluyente, que teóricamente proporcionaba un dato de caudal mucho más exacto, produce una retención de los flotantes en avenidas, en las que siempre circulan árboles de gran porte, que llegan a ocluir el paso del agua por la estación, distorsionando totalmente el registro de niveles y perdiéndose por esta causa un dato tan importante como el registro de la avenida de 100, 200, 500 … años de periodo de retorno. Figura 3 y Figura 4.

Actualmente se puede realizar una adecuada calibración de los datos de aforo mediante modelos hidrológicos e hidráulicos y nuevos sistemas

Figura 3. E.A. nº 154, río Tastavíns en Peñarroya. Antigua instalación.

Figura 4. E.A. nº 154, río Tastavíns en Peñarroya. Nueva instalación.

de aforo más sofisticados (como los perfiladores acústicos doppler) que permiten la toma de datos de caudal en situaciones extremas de avenida y proporcionan valores (altura-caudal) muy valiosos, y hasta ahora imposibles de obtener con precisión, para el ajuste de la parte alta de las curvas de gasto.

Las series validadas de datos foronómicos son la base de un buen estudio hidrológico de las aportaciones esperables en un emplazamiento seleccionado para construir un embalse, tanto en valores de volúmenes medios como en las series de máximos empleadas en el diseño de los órganos de desagüe y desvío del río en las distintas fases de la obra.

En la explotación normal y gestión de los embalses es deseable contar con estaciones de aforo que cuantifiquen las aportaciones en todos los cauces que confluyen en el vaso, así como en las salidas totales del caudal a pie de presa o en canales o derivaciones por tubería que salen de la presa.

Los Sistemas de Ayuda a la Decisión (SAD) conectados con los datos en tiempo real de los Sistemas Automáticos de Información Hidrológica (SAIH) permiten hacer previsiones de evolución de las variables de nivel y caudal en las estaciones de aforo a partir de la situación inicial de las mismas y la previsión meteorológica, que incorpora los datos necesarios (precipitación, temperatura) para que los modelos hidrológicos generen los caudales de escorrentía superficial (incluida la fusión nival que en las cuencas pirenaicas y en otras cuencas tiene un gran relevancia). El correcto ajuste de estos modelos depende directamente de la calidad de los datos proporcionados por las estaciones de aforo, que son las secciones de comprobación del caudal generado en cada una de las subcuencas hidrológicas en las que se divide el modelo.

En conclusión, un adecuado diseño de las estaciones de aforo, unido a un buen seguimiento y control de las mismas, genera buenos datos de nivel y, con una cuidada curva de gastos, se transforman en datos de caudal de calidad que dan como resultado un preciso dimensionamiento de las obras hidráulicas y una buena gestión del recurso hídrico en cualquier cuenca hidrográfica.

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Avería en la central hidroeléctrica y la seguridad hidrológica de la obra hidráulica de Sayano-Shushensk

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ABSTRACT: As a result of the tragic accident on the Sayano-Shushensk hydropower plant on 17 August last year, the water pass works of the project decreased considerably. The problem of the large and extreme floods evacuation with a part of turbines idling in inoperative condition and the faulty stilling pool of the existing spillway requiring the urgent constructive and operative measures. The description of real situation and the problems to be solved and numerical examples of the probable floods routing is the topic of presented report.

INFORMACION GENERAL

Como es sabido, el 17 de agosto del año pasado occurió la catástrofe en la central hidroeléctrica de Sayano-Shushensk en la que perdieron la vida 74 personas. A resultas del siniestro la planta eléctrica más potente de Siberia quedó fuera de servicio.

En cuanto a las causas de la avería y la física de este fenómeno hay diferentes opiniones. La mayoría de los expertos opinan que la destrucción de la sala de máquinas es resultado de la turbulencia inadmisiblemente grande en la cámara de turbina de la unidad №2 puesta en marcha en el año 1979. Esta turbina tenía zonas de potencia con alta vibración del apoyo de la quicionera y de la tapa de la turbina. Aunque estas zonas prohibidas estaban conocidas por el personal de la central, el día 17 de agosto los operadores del sistema energético de Siberia durante las horas de mañana varias veces cambiaron (aumentaron y bajaron) la carga eléctrica pasando las zonas de potencia peligrosas para la turbina.

La explotación ininterrumpida durante varios meses del grupo №2 con la vibración elevada provocó los detrimentos de fatiga en las fijaciones y apoyos de la estructura y la ruptura de los pernos (d = 80 mm) que juntaban la tapa de la rueda de turbina con el anillo del estator. La inspección de los restos de la sala de máquinas después de la avería reveló la falta de tuercas en seis pernos.

La central hidroeléctrica de Sayano-Shushensk es el primer (superior) escalón de la cascada hidroeléctrica en el río Yeniséy. Las características principales de diseño de centrales mencionadas están presentadas en la Tabla 1.

Tabla 1.

Indices	Sayano-Shushensk	Mainskaya	Krasnoyarsk
Hidrología area de aportación, km^2	179900	180000	288200
Escurrimiento medio anual, km^3	46,7	47,0	88,0
Caudal máx. afluente*, km^3/s	24400	25000	39000
Embalse nivel normal, m	540 (539)	326	243
Nivel mínimo, m	500	319	225
Nivel máximo, m	544,5	326,5	2
Volumen total, km^3	31,3	0,12	73,3
Volumen útil, km^3	15,3	0,07	30,4
Obras hidráulicas Tipo de presa	Bóveda y de gravedad	De tierra	De gravedad
Altura máxima, m	245	31	124
Potencia instalada de la central, MW	640 × 10	107 × 3	500 × 9
Generación anual de energía, GWH	23500	1717	20400
Salto de diseño, m	124	17	93
Caudal de turbina, m^3/s	358	725	615
Caudal máximo evacuado, m^3/s	15900	12100	19380

*Con la probabilidad de excedencia 0,01%.

La planta de estructuras hidraúlicas, la vista desde aguas abajo y las secciones de la presa vertedera y de la presa con la central aparecen en las Figuras 1, 2, 3, 4.

En el proceso de explotación de la central hidroeléctrica se detectó que el cuenco amortiguador del vertedero que según el proyecto estaba calculado a la descarga máxima de 13000 m^3/s con la velocidad de flujo de 60 m/s (el caudal específico 121 m^3/s/metro lineal) en realidad no permitía evacuar más de 5000 m^3/s. El revestimiento del cuenco (bloques de hormigón con espesor de hasta 6 m) había sido destruido tres veces: durante las crecidas de los años 1981, 1985 y 1988. Por eso hace 3 años en la orilla derecha del río se comenzó la construcción del vertedero lateral con la capacidad de 4000 m^3/s.

El nuevo aliviadero tiene 2 túneles de 11 m de diámetro y la rápida de descarga (vertedero de 4 escalones) con canalones, bajas presas vertederas y cuencos amortiguadores.

Los detalles constructivos del aliviadero aparecen en las Figuras 5 y 6.

La puesta en operación del aliviadero había sido programada para el verano de 2011.

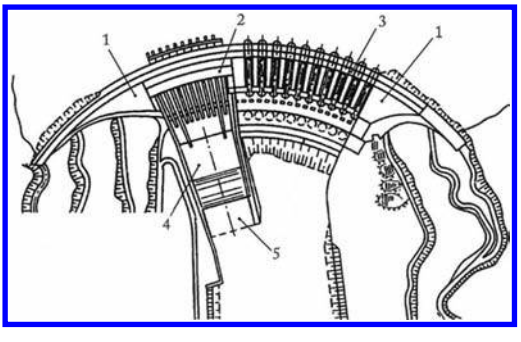

Figura 1. Planta de obras hidráulicas. 1. Presa. 2. Presa vertedera. 3. Presa con la central. 4. Cuenco amortiguador. 5. Zampeado.

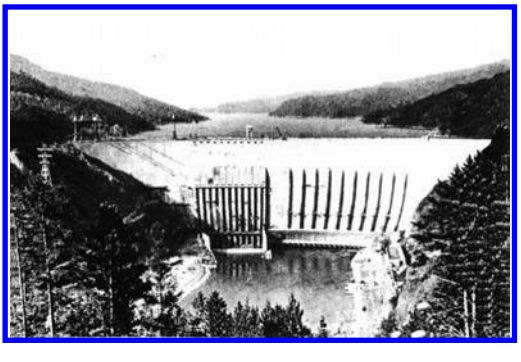

Figura 2. Vista desde aguas abajo.

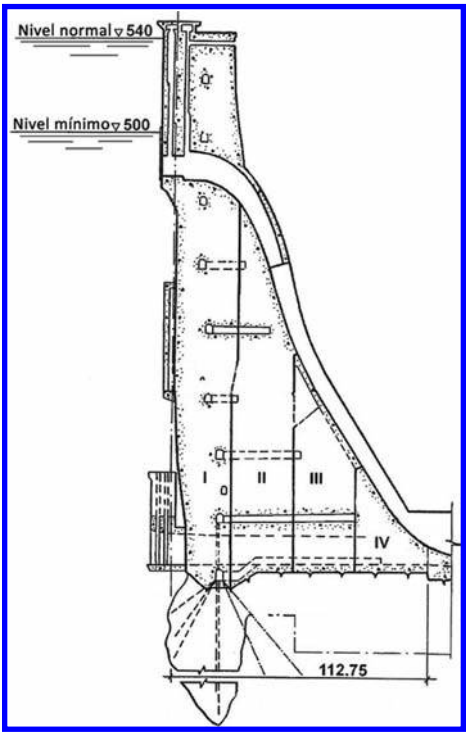

Figura 3. Sección de presa vertedera.

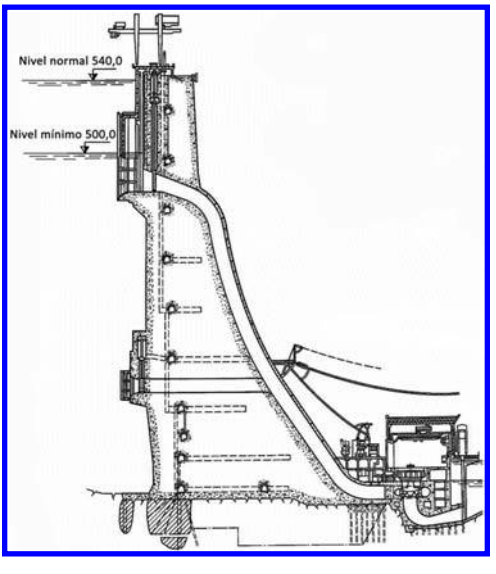

Figura 4. Sección de la presa con la casa de máquinas.

La avería del 17 agosto de 2009 cambió la situación y complicó seriamente las condiciones de evacuación de las crecidas.La primera dificultad era la necesidad de evacuación de los caudales

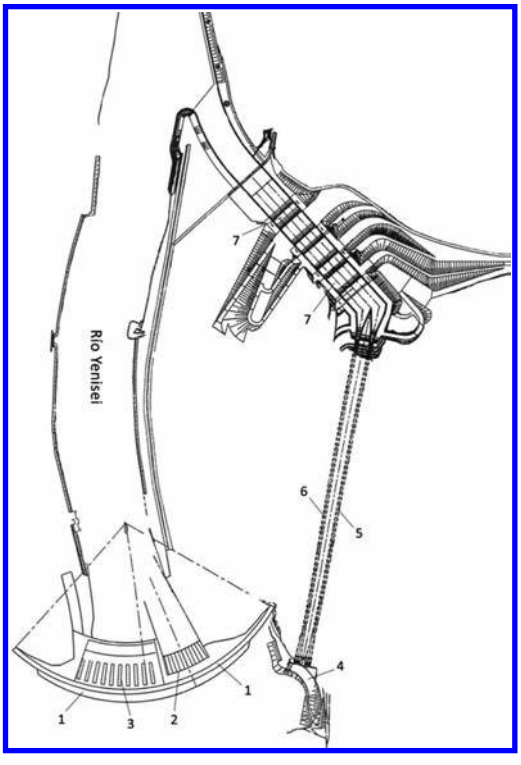

Figura 5. Planta de la obra hidráulica de Sayano-Shushensk con nuevo aliviadero. 1. Presa bóveda de gravedad. 2. Presa vertedera. 3. Presa con central hidroeléctrica. 4. Portal de aducción. 5. Túnel derecho. 6. Túnel izquerdo. 7. Cascada de cuencos amortiguadores.

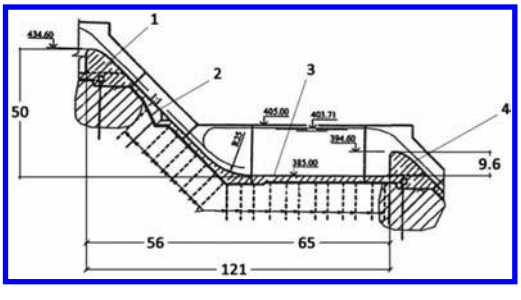

Figura 6. Eslabón de amortiguación. 1, 4. Presa. 2. Rápida. 3. Cuenco amortiguador.

durante el invierno: el aliviadero no estába destinado a funcionar con las temperaturas de aire de hasta 30°C bajo cero con la congelación de la lámina vertiente y con todas las turbinas fuera de servicio. Esta etapa de operación de la presa en condiciones postaveriadas pasó sin sopresas desagradables. Dos unidades fueron reparadas y reinstaladas en marzo y una en agosto 2010 y hasta el momento funcionan normalmente. En 6 meses de operación la central generará 2000 GWH de energía eléctrica.

A partir del 1 de octubre del año pasado se está construyendo urgentemente uno de dos túneles con su cascada de presitas y cuencos amortiguadores. Se suponía que esta mitad del aliviadero adicional estaría lista para dejar pasar los caudales de crecida en el verano de 2010. Felizmente la capa de nieve sobre el area de aportación no era grande y por eso se pronosticaba (y fue así) una crecida moderada.

Si este verano las condiciones del paso de la crecida a través de la obra hidráulica dañada fueron favorables, las posibilidades de evacuación de la crecida de diseño y la de seguridad en el futuro son mucho más complicadas.

Según las normas nacionales la obra hidráulica de Sayano-Shushensk con la presa de 240 m de altura es una construcción de primera clase que tiene que ser calculada a la evacuación de la crecida con probabilidad de excedencia igual a 1/1000 y chequeada para la de 1/10.000. La solidez de la presa determina la seguridad de la población en el valle de Yenisey aguas abajo de la central donde se sitúan las ciudades de Minusinsk (80.000 habitantes) y Abacán, capital de la república de Jakasia (160.000 habitantes).

Durante más de 30 años de su explotación la obra hidráulica de Sayano-Shushensk sufrió varias desgracias. Además de los deterioros del cuenco amortiguador arriba mencionados después de unos años de operación se revelaron las grietas en el paramento aguas arriba de la presa y la filtración excesiva de agua a través del cuerpo de la presa. En cooperación con la empresa SOLETANCHE estas grietas habían sido enmasilladas y la filtración fue reducida hasta valores admisibles. Pero con la subida regular (cada verano) del nivel de agua en el embalse la apertura de las grietas aumenta y por eso el nivel normal del embalse hace unos años bajó en un metro (hasta la cota de 539 m). Actualmente el nivel máximo de diseño (544,5 m) no puede exceder 540 m y es deseable que en las crecidas de rutina el nivel del embalse no exceda la cota de 536 m.

Para cumplir con esas restricciones las compuertas del aliviadero de explotación (caudal vertido hasta 5000 m^3/s) tienen que abrirse en el comienzo de la crecida con el nivel de embalse cercano al mínimo aunque esto en el caso del pronóstico equivocado puede llevar al llenado incompleto del embalse.

Es evidente que hasta la rehabilitación completa de la obra hidráulica (a finales de este año se supone poner en marcha cuarta unidad) incluso si el aliviadero lateral queda completamente construido, el proyecto de Sayano-Shushensk es capaz

de dejar pasar sin complicaciones las avenidas con la probabilidad de excedencia de más de un 1% (período de retorno 100 años). Las características de crecidas de diferente probabilidad de excedencia (período de retorno) y posibilidades de su evacuación aparecen en la Tabla 2.

Como se ve de la tabla 2, la capacidad reguladora del embalse es muy modesta y constituye alrededor del 60–70% del volumen de la onda de crecida de 30 días y la suma de caudales que pueden pasar através de las obras de desagüe (abredeor de 11 000 m^3/s) es escencialmente menor que el caudad máximo afluente con período de retorno de 10000 años (23 900 m^3/s) y relativamente cercano al maximum de 100 años de retorno (13 500 m^3/s).

Los resultados de cómputos del paso de crecidas con los períodos de retorno de 100 y 10 000 años en condiciones arriba descritas están en la Fig. 7.

Como se ve en la Fig. 7, la capacidad de desagüe actual de la obra hidráulica es suficiente para dejar pasar la crecida centenaria. La cota de nivel de agua máximo es igual a 533 m lo que es muy favorable para la presa.

Tabla 2.

Indice	Valor		
Avenida			
Probabilidad de excedencia, %	0,01	0,1	1
Caudad máximo, m^3/s	23900	17600	1300
Volumen, km^3, 20 días	27,6	20,5	16,0
30 días	34,1	25,9	20,9
Embalse			
Volumen útil, km^3			
Al nivel 535 m	12,4		
Al nivel 539 m	14,7		
Al nivel 540 m	15,3		
Capacidad de desagüe			
Aliviadero existente. Cota del umbral, m	479		
Caudal máximo admisible, m^3/s	5000		
Nuevo aliviadero. Cota del umbral, m	524		
Caudal máximo, m^3/s, con nivel de agua 535 m	2780		
539 m	3900		
540 m	4300		
Caudal de turbinas, m^3/s			
4 turbinas	1400		
6 turbinas	2100		
Con 6 turbinas	9880		
Caudal total al nivel 539 m			
Con 4 turbinas	10300		
Con 6 turbinas	11000		
Caudal total al nivel 540 m			
Con 4 turbinas	10700		
Con 6 turbinas	11400		

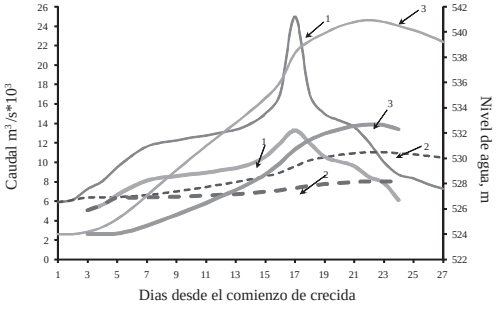

Figura 7. Transformación de crecidas con período de retorno de 100 años (líneas gruesas) y 10 000 años (líneas finas). 1. Caudal afluente; 2. Caudal vertido; 3. Nivel de agua.

La crecida de diez mil años (caudal afluente 24 000 m^3/s) lleva a la subida de nivel de agua hasta la cota de 541,4 m y las condiciones críticas del estado de la presa.

Las consideraciones y resultados de cómputos antes citados caracterizan la situación probable durante los próximos años. En un futuro lejano cuando todas las turbinas queden reparadas o reemplazadas por otras nuevas el problema de evacuación de crecidas excepcionales lamentablemente será bastante serio.

Formalmente, según el diseño, las 10 turbinas de la central son capaces de dejar pasar un caudal de 3600 m^3/s. Pero hasta el momento la capacidad instalada de la central ni una sola vez ha sido aprovechada ya que la línea de alta tensión puede transmitir la carga eléctrica de 4200 MW. Así, en la evacuación de crecidas pueden participar solamente 6 turbinas.

En caso de que llegue la crecida de seguridad con período de retorno de 10 mil años la dirección de la central hidroeléctrica se verá obligada a abrir las compuertas de todos los vanos del vertedero de explotación.

El deterioro inevitable del cuenco amortiguador que soporta solo los caudales menores de 5000 m^3/s sera el pago por la conservación de la obra hidráulica.

Sin embargo según los cómputos recientes la crecida de seguridad en condiciones arriba formuladas con la abertura de las compuertas a la cota de agua de 524 m en el caso del pronóstico de avenida extraordinaria puede ser evacuada sin exceder la cota máxima del embalse de 540 m.

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Antifiltration grout curtain of hydro power plant Piva—Mratinje dam

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ABSTRACT: Mratinje dam site is located in the north of Montenegro on the Piva river, at the chainnage of km 10 + 600 from the Piva confluence with the Drina river. The dam site was named after the nearby Mratinjska Reka river, an upstream tributary of the Piva river. The wide area of the dam site is composed of Triassic limestones and dolomitic limestones of reef origin. Considerable fissuring of the limestone rock mass as well as carbonate contents of the these rocks, enabled karstic erosion to develop in this area. Experimentaly determinated water-permeability is in the range of 1–4 Lu i.e. the average water-permeability was defined as 2.14 do 4.86E-7 m/sec with the effective porosity of E = 0.0021–0.0005. In the conditions of construction, a grout curtain was designed and constructed in both dam abutments for the purpose of ensuring the designed conditions regarding seepage and allowable groundwater levels in the downstream part of the dam. Grout curtain was constructed from the tunnel at the elevation of 680. I the left abutment a three-row curtain was constructed in the length of 55 m. A single-row curtain constructed in the length of 300 m stretches below the three-row curtain. In the right abutment, a three-row curtain, which was constructed in a subsequent intervention in 1981, is 95 m long and the length of a single-row curtain is 360 m, which gives the total of around 455 m. In the right abutment a drainage curtain was designed and constructed from the grouting gallery at the elevation of 562 m for the purpose of ensuring further lowering of the groundwater level downstream from the grout curtain. Monitoring network consists of 56 piezometers arranged in both dam abutments. Outflows downstream from the dam, which are measured on regular basis, amount to 70 to 110 l/sec in total. Within the project of HPP Piva rehabilitation at the end of 2009, hydrogeological investigations of the grout curtain were performed. They included groundwater tracing test, measurements of outflows from the drainage gallery and at all downstream outflow points, measurements of downstream Piva river discharges and analyses of chemical water contents. Facts and analysis of investigation results, conclusions and proposals related to the grout curtain revitalization are presented in this paper.

1 INTRODUCTION

The paper contains a brief overview and preliminary analysis and assessment of geological and hydrogeological conditions of the grout curtain in Piva HPP, as well as proposals related to the modernization and revitalization.

The first part of this paper contains a brief overview of the composition and properties of the geological medium—ground, as well as of the main hydrogeological properties of the ground in the structure impact area. It is followed by a description of the ground state with the focus on its hydrogeological properties after construction, during and after filling of the reservoir, including assessment of grout curtain functioning. The Report also contains a detailed presentation of all subsequent repair works executed in the zone of the right dam abutment, as well as results of tests performed in that period.

The groundwater level in the dam abutments is first of all dictated by fluctuations in the reservoir. Besides the reservoir, the grout curtain, the drainage structures, as well as the tailwater level elevation have significant influence on the basic aquifer regime. Certainly, the position and the regime of the ground waters is generally conditioned by the geological composition of the terrain, by the hydrogeological properties of the rock masses, and within this, by the regime of the ground waters in the hinterland of the reservoir, as well as by the precipitations regime.

Groundwater level in dam abutments is monitored by means of 56 piezometers. The main

database contains results of groundwater level observations by piezometers, measurements of water levels in the reservoir and data about precipitation in the observation period, as well as measurements of discharge at the exit from the drainage gallery at the elevation of 522 asl. and Bailey bridge source yield at the right abutment.

2 GENERAL HYDROGEOLOGICAL CHARACTERISTICS

Wider area of the dam site has been built from the limestones and the dolomite limestones of the reef origin. These are the massive limestones with rare occurrence of beds, their colour ranges from grey to whitish. The sedimentological investigations showed that the dolomite limestones are predominantly present, sometimes they are precrystalized with the content of irregular dolomite crystals. The carbonate substance is usually irregularly distributed.

The significant jointing of the limestone rock mass and the presence of several fracture systems (out of which the most interesting are the systems that cross-cut the Piva river valley in the northwest—southeast and northeast—southwest directions), as well as the carbonate composition of these rocks, enabled very favourable development of the karst erosion in these terrains. This process

Figure 1. Location of the HPP Piva.

especially progressed along larger breaks and faults (and the other mechanical separations in the rock massif) and it was conditioned by very deep terminant of the river drain and by high gradients; therefore it lowered significantly below the present riverbed.

The basic hydrogeological characteristic of karst in the wider area of the dam site reflects in the presence of basic karst phenomena, such as: large springs, springs, periodic springs, caves, rock shelters and other similar forms that are encountered in the karst.

Comparing the left river bank with the right one, it should be underlined that there is a significantly higher number of lage springs and springs on the right river bank and therefore richer resources and higher discharge of ground waters from the hinterland at the right riverbank than from the hinterland at the left riverbank. This phenomenon is interpreted on the one side by the geological composition of the terrain and on the other side by the catchment area. The proximity of the Mratinje anticline in the southwest direction and small river basin in the hinterland of the dam, contribute to lower inflow of waters from the hinterland, and thereby to the less prominent karstification, i.e. the level of karstification base is higher in the left abutment.

On the other side, the favourable structural composition of the terrain, which conditioned displacement of the hydrogeological water divide between the Piva river and the Tara river, to the detriment of the Tara river; turning of the apex part of the Crkvica anticline towards the southeast and larger catchment area, as well as higher precipitations, contributed to the fact that the karstification process is significantly more developed on the right side of the valley than on its left side, therefore the karstification basis on the right bank is much more lowered.

This situation related to asymmetrical development of the karst erosion, has its undoubted confirmation in the differences of the gradients which were established by drilling and by observing of the piesometric pressures in the dam site area. Namely, in the natural conditions significantly higher gradients of the ground waters were established in the hinterland of the left bank (0.3) than on the right bank, where the gradient at the observed profile amounted to 0.02.

3 ROCK MASS PERMEABILITY

For the purpose of defining properties of the rock mass, it was established during the investigations that water-permeability in the grout curtain zone is low in natural conditions and that it varies in the range of 0.1 to 5 Lu. Such water-permeability leads

to the conclusion that cracks with small openings ranging between 0.002 and 2 mm are predominant in the massif and that fractures exceeding 5 mm are rarely encountered, which points to fine fissuring of the rock mass. The rock mass with such characteristics is hardly groutable. Based on the analysis of results obtained by WPT tests, the coefficient of 2.14 to 4.86*10^{-7} m/sec was obtained wherein the effective porosity amounted to E = 0.0021–0.0005. Tests were also performed in piezometers and they produced the following results related to water-permeability characteristics:

Piezometer P-10 k = 2.24*10^{-7} m/sec E = 5*10^{-4}
Piezometer P-11 k = 5.16*10^{-7} m/sec E = 0.0021
Piezometer P-12 k = 6.7*10^{-6} m/sec E = 5*10^{-4}

The calculated porosity value amounts to around 0.1%.

4 GROUT CURTAIN

4.1 *Grout curtain elements*

The grout curtain was designed and constructed in both abutments in conditions of construction of the structures, for the purpose of establishing the design conditions, from the point of view of seepage and permitted levels of the ground water in the downstream part of the dam.

The grout curtain in the left abutment was constructed from the tunnel at the elevation of 680 m asl. The three-row grout curtain was constructed in the length of 55 m. At the beginning of the tunnel the curtain was constructed up to the elevation of 522 m asl, whereas at the end of the tunnel the curtain finishes at the elevation of 604 m asl. The single-row curtain was constructed in the length of 300 m, and it also included a part underneath the three-row grout curtain. In the abutment, immediately along the river, it reaches the elevation of 350 m asl, then its depth reduces and at the end of the three-row curtain it reaches the elevation of 480 m asl. The single-row curtain, approx. 250 m long, finishes at the elevation of 480 m asl. Total depth of the single-row grout curtain amounts to approx. 200 m and it was constructed from one level only, i.e. from the grouting gallery—from the elevation of 678 m asl. From the same gallery the transverse overhangs (15 m long) were constructed downstream of the curtain, at the end of which the piezometers for the hydrogeology oscultations were constructed. Two overhangs (each of them 10 m long) were constructed upstream of the curtain, with piezometers for observation of the ground water level upstream of the curtain.

The grouting gallery, approx. 480 m long, was constructed in the right abutment, at the elevation of 680 m asl and the grout curtain was constructed from the grouting gallery. The three-row grout curtain was constructed in the length of 95 m, up to the level of the piezometer P-11, whereas the single-row curtain—360 m long was constructed approx. 30 m behind P-15 piezometer, meaning that the total length of the curtain amounts to approx. 455 m. The mutual distance between the boreholes in the three-row curtain amounts to 3 m. The depth of the three-row curtain ranges from 168–190 m, whereas the depth of single-row curtain ranges from 175–245 m.

Analysis of available data dating back to the time of construction showed that the average consumption of dry substance had not been large.

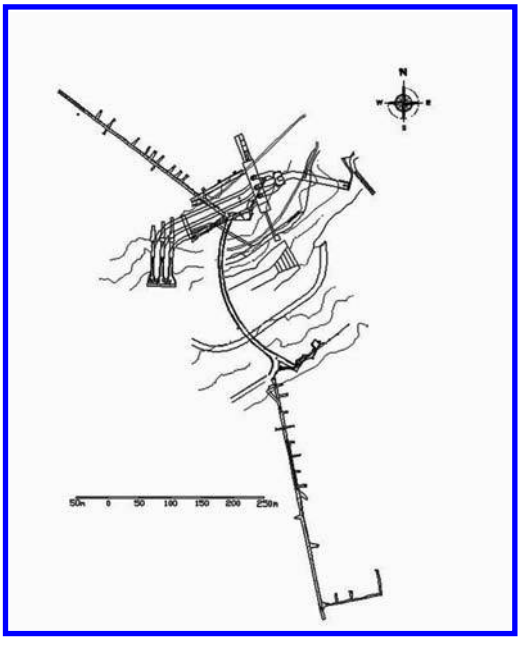

Figure 2. Position of the dam and grouting galleries.

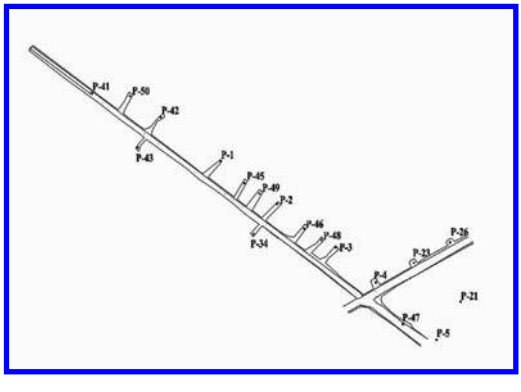

Figure 3. Left abutment grouting gallery.

Consumption of dry substance was large only in certain grouting sections, at various levels. These data, as well as results of other investigations, show that the rock mass is relatively poorly permeable. Grouting was performed by applying the downward method. In cases of small consumption, water-cement paste (cement-water ratio 1:4) with addition of 5% bentonite was used, combining various ratios of water-dry substance. In cases of large losses, sand was added to form concrete in large fractures and they were subsequently grouted using cement. The grouting was performed in 5 m sections with 3 m interval of boreholes. For subsequent additional repair grouting performed in the zone of three-row curtain in the right abutment, the interval of grouting boreholes was reduced down to 1 m. Grouting was defined as successful in cases when the result of WPT in control boreholes was below 1 Lugeon (1 l/min/m/10 bar).

4.1.1 *Ground water regime in natural conditions before forming of reservoir*

4.1.1.1 Left abutment

Measurements in the left abutment were performed (June 1972 to October 1975) on the following piezometers immediately downstream of route of the grout curtain: P-5, P-4, P-1, P-2 and P-3, as well as on the following piezometers downstream of the curtain: P-22, P-26, P-30 and P-31. Fluctuations of the ground water levels in this period generally are in compliance with the precipitations.

If the amplitudes of the piezometric level fluctuations in the boreholes P-22, P-4, P-3, P-2 and P-1, which are located in the profile immediately downstream of the curtain, are considered, it can be noticed that the fluctuations in the borehole P-22, which is the closest to the river, range from 2 to 6 m. The amplitudes in the next two piezometers P-3 and P-4 range from 15 to 25 m, whereas in the most remote piezometers—P-1 and P-2 the amplitudes range from 2 to 10 m. The amplitudes decline with increasing of the distance from the river.

Table 1. Extreme values and amplitudes of groundwater fluctuations.

	1973			1974		
Piezometer	min	max	ΔH(m)	min	max	ΔH(m)
P-4	540.0	562.3	22.3	551.1	–	–
P-3	574.2	590.1	15.9	578.3	598.5	20.2
P-2	570.1	580.3	10.2	567.4	570.3	2.9
P-1	568.3	575.5	7.2	566.8	574.3	8.2
P-26	544.5	552.2	7.7	529.2	552.5	23.3
P-30	514.5	529.5	15	–	–	–
P-22	508.2	514	5.8	494.5	496.6	2.1

After analyzing the fluctuations in the piezometers, it can be concluded that the zone between P-4 and P-3 has lower water permeability than the zone between the river and P-22, as well as the zones between the piezometers P-2 and P-1.

As it can be seen, the piezometric level was high above the river level. During the maximum water level it amounted up to 90 m, whereas during the minimum water level it amounted up to 80 m above the river level.

4.1.1.2 Right abutment

On the basis of the observations of the piezometers in the right abutment during the same period of time, the minimum levels occur in the different time intervals whereas the maximum level predominantly occurred in October 1974. The minimum levels ranged from 514 m (P-14) up to 530.7 m (P-13). The maximum levels can be seen in the following table:

Table 2. Extreme values and amplitudes of groundwater fluctuations.

Piezometer	min	max	ΔH(m)
P-35	513	554.2	41.2
P-36	513.4	608.3	94.9
P-15	612.6	630.3	17.7
P-14	504	595	91
P-13	530.7	550	19.3
P-12	512.5	561.4	48.9
P-11	516.3	565	48.7
P-28	490.1	502.6	12.5
P-29	509	529.1	20.1
P-23	485	503	18
P-24	502	518	16
P-25	520	559	39
P-7	452.5	499	46.52
P-10	517	596	79
P-9	511	550	39

The extreme levels in the conditions of intensive precipitations were significantly higher, thus the amplitudes of water level fluctuations ranged from 20 up to even 94 m. Such fluctuations indicate that the periodical inflows of ground water are significantly higher than the permeable capacity of the massif, thus this ground water is temporarily stored in the rock massif and then, after the precipitations stop, the ground water seeps.

4.1.2 *Ground waters regime after forming of the reservoir and assessment of the grout curtain effects*

4.1.2.1 Left abutment

The hydrogeological observation network in the left abutment comprises 18 piezometers. Because of very unfavorable topography, it was not possible

to place the optimum observation network. The tunnels for the grout curtain, as well as the road, were used for construction of the piezometers.

The piezometers P-1, P-2, P-3, P-34, P-42, P-43, P-45 and P-46 have been observed for the longest period of time. Since 1987 the renovated piezometers P-4, P-23 and P-26 were included in the observation network and since 1990 and 1991 the piezometers P-47, P-48, P-49, P-50 and P-21 have also been included in the observation network.

The ground water regime upstream of the grout curtain is controlled by P-34 and P-43 piezometers. Regime of the ground water levels at the end of the curtain is controlled by P-41 piezometer, whereas the rest of the piezometers control the fluctuations of the ground water levels downstream of the curtain.

Filling of the reservoir with water started in October 1975. In accordance with the observations and conclusions from the time of filling of the reservoir with water, the water level elevation in the reservoir up to the elevation of 570 m asl, practically has no influence on the level of the ground water in the piezometers P-1, P-2 and P-3. The influence of the reservoir to increasing of the water level in the above mentioned piezometers downstream of the curtain occurs only with the water level elevation above this level.

Influence of the reservoir to the ground water level downstream of the curtain is variable, with the tendency of rising practically at all piezometers, but only with different amplitudes. The Table 3 presents the values of water levels in the observed piezometers, for the characteristic maximum values of the reservoir levels, for the different dates.

Table 3. Piezometric levels under conditions of maximum water levels in reservoir—left abutment.

Piezometer	5/7/01	5/7/04	12/7/04	6/3/05	7/4/06
P-1	605.56	607.61	607.71	605.86	604.76
P-2	630.87	637.47	636.82	627.02	623.97
P-3	617.21	617.81	614.16	613.86	612.26
P-4	603.66	604.51	602.96	603.11	604.71
P-5	524.59	526.34	527.44	527.44	526.74
P-21	537.15	546.1	541.95	539.3	539.8
P-23	553.84	572.84	563.99	554.14	550.84
P-26	539.00	555.65	553.10	542.60	538.40
P-34	672.71	676.16	674.01	674.41	674.56
P-41	652.97	657.77	653.57	651.47	651.42
P-42	619.33	625.28	615.58	616.58	615.78
P-43	656.84	662.39	659.29	655.54	655.44
P-45	614.07	632.47	642.87	613.62	610.57
P-46	632.70	634.00	632.80	631.70	631.60
P-47	608.08	610.28	609.98	608.58	610.93
P-48	637.28	638.53	637.23	636.08	636.68
P-49	603.16	609.01	609.46	604.96	601.11
P-50	608.31	608.75	607.75	605.65	606.10
Reservoir	672.92	675.25	674.12	674.30	674.94

The piezometer P-34, located at 150 m from the reservoir rim, completely complies with the changes of water levels in the reservoirs, without any delay. At that section next to the curtain, the ground water level is practically identical to the level of water in the reservoir. The levels of ground water in the piezometers P-43 and P-41 are under stronger influence of the precipitations, which reflects on their fluctuations. In accordance with the recorded measurements, it is evident that the gradient from P-34 towards P-43 is higher, whereas from P-43 towards P-41 the gradient maximally reduces, which also depends on the regime of the water levels in the reservoir.

Main task of the analyses of levels and regimes of ground water in the downstream piezometers in comparison with those in the upstream piezometers, as well as with the level in the reservoir, is to render the data and to assess the watertightness of the dam site profile. If necessary, on the basis of this information the proposals of necessary measures for providing the design watertightness criteria can be given. Generally, the downstream piezometers less reflect the influence of the reservoir and they more reflect the drainage effects which occur under the influence of the rock mass, the tail water level, as well as the precipitations.

A group of piezometers P-5, P-21, P-23, P-26, P-47 and P-4 is located in the zone of the dam support and therefore they have special significance. The riverbed and the structures constructed in the zone of left abutment have the dominant influence on the levels of ground water in the piezometers P-5, P-21, P-23 and P-26, thus the influence of reservoir on the levels of ground water in these piezometers is insignificant.

The piezometers P-47 and P-4 are located in the sensitive zone of the contact between the dam and the rock mass and they record relatively high ground water levels. Therefore the attention should be devoted to this phenomenon in the next stage of designing. This procedure should comprise redefining of limit values of water levels in the control piezometers. Also, it is necessary to include the piezometer P-47 into this group of piezometers, having in mind that it is located in the zone of abutment and that the observations in this piezometer have special significance.

The piezometers in the central part of the curtain: P-1, P-45 and P-49 have lower water levels than the piezometers located towards the dam, i.e. in the hinterland and they show somewhat better effects of the grout curtain. Relatively high levels were recorded in the stretch from the piezometer P-2 up to P-47, i.e. in the piezometers P-2, P-46, P-48, P-3, P-4 and P-47. Therefore, it can be confirmed

that there are clear indications that the first part of the grout curtain in the left abutment has weaker effects. In accordance with the conclusions of the investigations of the effects of the curtain from the period of filling of the reservoir with water, its effect is weaker only in its part from the borehole P-46 and P-3. This higher increment of level in the massif is interpreted by weak water permeability of the rock mass; therefore the full effects of the grouting works have not been achieved.

The relation between the measured limit values and the limit values defined through calculations of piezometric levels for the control piezometers is such that the exceedances constantly occur in the piezometers P-2, P-3 and P-4 and the same order of magnitude of exceedances occur for the entire period of observation. Such conclusion requires redefining and recalculation of limit values i.e. verification of influence of high piezometric level to the dam stability. At the same time all elements of the natural environment, influences on the dam and the previous functioning of the structures should be taken into consideration. The proposals of possible remedy measures, within the final documentation of the Study, depend on these results.

However, it should be born in mind that in previous functioning of the grout curtain, which was analyzed during preparation of the Annual Reports and the analyses of the piezometers observations, no deteriorations of the existing piesometric levels have been registered, i.e. no visible subsequent degradation of the grout curtain has been recorded. If on the basis of repeated calculations of the limit values the conclusions concerning necessity of undertaking of the remedy measures would be brought, these measures would probably be directed to construction of the drain curtain downstream of the grout curtain, exactly because of poor water permeability of the massif, which caused high natural level of the ground water.

The analysis of linear trend of water levels in the observed piezometers in the period 1999–2007 does not show the elements of additional degradation of the curtain, because the trend at all piezometers is less or equal to the trend of the reservoir level.

4.1.2.2 Right abutment

New water level in the hill massif has formed and the permanent change of hydrogeological conditions in the massif, upstream of the dam, has occurred during filling of the reservoir with water. During filling of the reservoir with water, this water percolates into the hill massif, raising generally the level of karst aquifer. The percolation lasts until the balance between the reservoir level and the inflow of water from the hinterland of the hill massif, establishes. The level of newly established aquifer fluctuates depending on the changes of the water levels in the reservoir. If the reservoir is maintained at the same level for a long period of time, the equilibrium between the water level in the reservoir and the ground water inflow establishes.

Very large differences of the levels of water in the massif downstream and upstream of the dam occur in the immediate vicinity of the dam—in the abutments. Width of the front of ground water flow spreads deeply into the massif, up to the point where the gradients of flow from the direction of the reservoir equalize with the gradients of the water inflow from the massif. A sudden bending of the aquifer, in the direction of discharging towards the Piva river valley, is formed at the dam level. The curtain which purpose is to form a sudden drop of downstream water level in relation to the upstream water level, is not capable of preventing the seepage and by-passing of water around itself, because it ends before equalizing of the water level elevation in the reservoir and the natural water level.

Water permeability investigations showed that the limestone massif has poor water permeability features. The predominant water permeabilities range from 0.1 to 5.0 Lu, whereas the intervals ranging from 5 to 10 Lu and from 10 to 20 Lu, occur only sporadically in some of the boreholes.

It was established through measurements that the capacity of Kaluđerovo vrelo large spring does not change under the influence of reservoir. In order to define these quantities, control

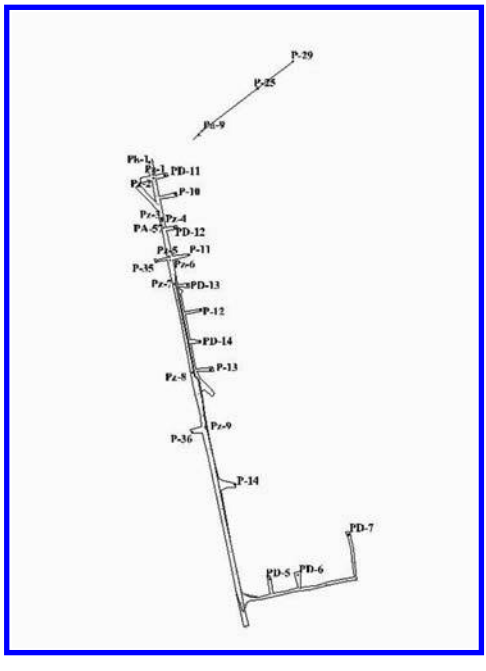

Figure 4. Right abutment grouting gallery.

hydrological measurements should be established at two sections, underneath the dam and above Kaludjerovo Vrelo spring.

During filling-in of the reservoir the level of water in the hill massif immediately increased, with variable differences between the water level in the reservoir and the water level in the piezometers. Practically there wasn't any difference in some piezometers at the water level elevation amounting to 639.9 m asl, whereas the significant difference (52.3 m) was recorded only in piezometer P-10.

In the period 1976/1977, fifteen (15) drainage boreholes were drilled in the fan-shape arrangement from the gallery at the elevation of 522 m (length of each borehole is 230 m). This drainage partially lowered the level of ground water in the massif and it also caused lowering of the water level in the piezometers P-13 (15 m), P-12 (43 m), P-11 (61 m) and P-10 (118.7 m), whereas in the piezometers P-14 and P-15 the change in relation to the reservoir level almost was not recorded, as well as in the piezometer P-9. The drainage obviously reduced the piesometric levels, although in majority of the piezometers these levels remained high above the natural maximum level.

It was noticed in the period immediately after filling-in of the reservoir with water that the grout curtain does not function within the design values. The investigations showed that the water flows through the curtain in larger or less amount.

Also, the additional urgent measures intended for reducing of the ground water level were necessary in the area immediately next to the dam abutment, as well as in the zone of the access tunnel, towards the piles. For this purpose boring of additional boreholes in the KS grout curtain was performed and the drainage boreholes were drilled from the gallery at elevation of 562 m asl.

The access tunnel for construction of the piles in the right dam abutment causes a particular problem in functioning of the curtain in the right dam abutment. This tunnel passed through the curtain, with downstream inclination. This tunnel was not completely covered with concrete when it was closed, meaning that some of its parts were completely open for communication of the ground water. Even besides the subsequent remedy and grouting works in the zone of the access tunnel the problem related to the privileged flow directions remained. In other words the water continuously flows between the concrete and the rock and it supplies with water the massif downstream the curtain, towards the zone of the piezometer P-9, which slows down draining of the zone between the dam and the piezometer P-10, as well as draining of deeper part of the massif, between the piezometers P-10 and P-15. This is the basic explanation for relatively high ground water levels in the zone downstream of the grout curtain.

For the purpose of recognizing and monitoring the effects of additional protection measures, the piezometric network was supplemented with a large number of boreholes, thus the existing observation network comprises 38 piezometers, including also the piezometers Pz-1, Pz-9 and PA-57, which have been included in the observation network in 1998 and are located in the grout curtain.

The regular observations of these piezometers give basic information on behaviour of natural environment in conditions of fluctuations of the reservoir level, from the point of view of functioning and effects of grout curtain, as well as fulfillment of the criteria of stability of the right dam abutment. The analyses of regular observations have been performed on the annual level since filling of the reservoir with water. On basis of the basic parameters and the results of these analyses, it can be said that the conditions in the zone of grout curtain have been slightly changing and that the assessments of hydrogeological investigations from the period of beginning of functioning of the structures, are still current.

Thus, effects of the single-row grout curtain, from the piezometer PD-14 up to its end point, can not be noticed and they result in significant seepage runoff through this part of the curtain.

The reliable conclusions related to the accomplished delevelling effects of three-row grout curtain can not be made only on the basis of observations of the piezometers, because it is mainly a result of functioning of drainage structures in the right abutment. However, after analyzing all the available data related to the current observations, as well as the data from the previous investigations, the certain sections of the three-row grout curtain have been determined as the sections with poor water tightness, which first of all refers to the zone of the piezometers from PA-57 up to Pz-7 (tracing investigations). The curtain as a whole does not function in compliance with the design parameters.

Influence of the reservoir is also recorded on the most downstream piezometers in the right abutment and this influence is somewhat stronger in the piezometer P-29 which is under smaller influence of the drainage structures and the tail water.

Analysis of linear trend of water levels in the observed piezometers for the period 2000–2007 does not show the elements of additional degradation of the curtain with regard to the fact that the trend in all piezometers is lower or equal to the trend of the reservoir levels.

The criterion of stability for the right abutment from the point of view of maximum allowed piezometric levels is given in the Study on notification and alert procedures (1989). The piezometers P-24

Table 4. Piezometric levels under conditions of maximum water levels in reservoir—right abutment.

Piezometer	5/7/01	5/8/04	12/9/04	6/6/05	7/3/06
P-9	604.84	609.94	609.79	602.19	602.54
P-10	–	548.81	548.11	549.56	–
P-11	608.30	621.60	613.30	611.65	611.85
P-12	626.91	636.21	630.56	628.91	628.66
P-13	658.41	665.51	662.56	659.96	660.11
P-14	661.26	671.86	668.21	666.21	665.76
P-15	666.33	674.48	670.53	667.68	667.33
P-25	543.30	549.05	544.55	545.75	545.85
P-29	565.54	574.54	567.49	557.44	567.79
P-35	673.03	675.03	674.28	674.28	674.63
P-36	665.83	669.63	668.83	666.63	670.73
PD-1	625.2	623.60	622.95	623.43	625.40
PD-3	580.42	580.52	580.47	581.72	581.12
PD-5	664.98	674.03	669.13	666.43	665.73
PD-6	665.32	674.77	669.37	666.52	666.12
PD-7	663.76	674.46	668.86	666.31	666.01
PD-12	611.23	615.38	614.98	612.23	612.18
PD-13	623.48	632.28	625.88	623.38	622.98
PD-14	633.75	642.95	638.05	635.75	635.65
Reservoir	672.85	675.20	674.46	674.38	674.84

and P-28 have not been observed. The recorded maximum levels in all other structures are lower than the allowed limit values. It would be necessary to establish the limit values for the allowed levels of ground water also for those piezometers which did not exist in 1989 and which are located in the zone of contact between the dam and the rock mass.

5 REPAIR MEASURES IN RIGHT DAM ABUTMENT

At the time of Piva HPP reservoir filling, it was established that piezometric level in the right dam abutment rose in parallel with the water level in the reservoir. With the water level of 640 m in the reservoir, water level in piezometers P-13, P-14 and P-15 was lower than water level in the reservoir by 3–6 m, in P-12 it was lower by around 15 m, in P-11 by around 27 m, in P-10 by around 53 m and in P-9 immediately by the dam it was lower by around 34 m. Behaviour of piezometric level in the hilly massif downstream from the curtain pointed to inefficient functioning of the curtain. Based on additional investigations, three fan-shaped drainage boreholes with various inclinations were drilled from the gallery at the elevation of 522 asl.

The total of 15 boreholes, 230 m each, was drilled. The drilling was completed in March 1977. After completion of the works, the discharge amounted to 100–120 l/sec, while after the initial rock mass drainage it stabilized at 60–80 l/sec. Yield measurements performed so far show that the quantity of water drained by means of this borehole system has not changed and that it mostly amounts to 60–80 l/sec, except in the rainfall period when it increases up to 90 l/sec as a result of inflow from the hinterland. The difference between water levels in the reservoir and in piezometers after drainage construction was as follows: P-15 (2 m), P-14 (9 m), P-13 (18 m), P12 (44 m), P-11 (58 m), P-10 (116 m) and P-9 (56 m). The effect of these drainage boreholes was most evident in the zone of piezometer P-10 immediately downstream from the curtain, while in some zones, particularly in P-14 and P-15, it was practically nonexistent.

Having in mind only partial effect of the drainage curtain from the gallery at the elevation of 522, curtain permeability tests were continued with and they involved regular observations of piezometer network and tracing tests. In the course of these tests, it was concluded that the access tunnel used for the removal of material during excavation of pile galleries, which cut through the grout curtain, had not been properly sealed when pile concreting i.e. that the concrete plug in the curtain zone had not been formed at all. This fact was identified as the main cause of high groundwater levels downstream from the curtain in the right dam abutment. For the purpose of preventing further impact of the reservoir on the downstream piezometric levels in the rock mass, a design of the access tunnel sealing in the zone of the grout curtain was prepared and implemented in two phases. The final grout works in this phase were executed at the beginning of 1981. Following access tunnel sealing and grouting, additional grouting of KS curtain was carried out and the curtain was transformed into a three-row one. The KS curtain was also connected to the three-row curtain in D gallery at the elevation of 678 m.

Based on the analysis of results of the repair measures undertaken for tunnel sealing and grouting works in that zone in the conditions of the maximum water level in the reservoir, it was concluded that water levels in the access tunnel zone were lower by 20 to 25 m compared to the water level in the reservoir, which was an obvious decrease, although the levels were still quite high.

For the purpose of ensuring further decrease of groundwater level downstream from the grout curtain, a drainage curtain from the grout curtain was designed and constructed at the elevation of 562 m. The total of 12 drainage boreholes was drilled in the length of 107 to 127 m, respectively. The boreholes were drilled in such a way to end between the elevations of 660 and 670 m asl. and they also drained the massif high above the access tunnel. The total discharge in all the boreholes did not exceed 1 l/sec. Such a small discharge points to general low permeability of this zone.

After construction of drainage boreholes at the beginning of May 1984, water levels in the reservoir remained above 670 m i.e. they were between 672 and 675 m in the period of four months. With these water levels in the reservoir, water levels in the access tunnel piezometers were lower by 30 to 40 m compared to the water levels in the reservoir. In this period large fluctuations and sudden changes of water level were not registered as a difference to some previous cases, which is interpreted as nonexistence of external impacts i.e. as a result of a period with extremely small precipitation. It was established that water levels in piezometers Pk-2 and Pk-4 were lower although water level in the reservoir was 50 m higher.

After drilling of drainage boreholes and draining of the access tunnel during the maximum water level in the reservoir, water level in the tunnel has been 30 do 40 m lower than water level in the reservoir, which is a result of repair measures. Analysis of individual effects showed that the effects of access tunnel grouting were larger and that they were in the range of 20–24 m while the effect of draining was 7 do 19 m.

6 LOSSES FROM RESERVOIR

Following reservoir filling, water emerged in the periodic Bailey bridge spring, the yield of which has been measured since then and varied in the range of 13 to 19 l/sec. The yield of this spring has not changed i.e. it has not gradually increased. It was concluded already in the phase of construction that losses from the reservoir were small.

After drilling of drainage boreholes from the gallery at the elevation of 522 asl., water seepage from the right side of the reservoir showed visible increase. A seepage flow measurement system was implemented and measurements of seepage from the gallery during various water levels in the reservoir were organized. These measurements have been performed constantly so far. At the beginning of operation, the discharge of the drainage gallery was 130 l/sec but later it decreased. The discharge of 60.6 l/sec with the water level of 634 m in the reservoir and 139 l/sec with the water level of 662 m in the reservoir was registered. Apart from that, emergence of a spring in the Piva riverbed downstream from the Bailey bridge was registered but direct measurements of the spring cannot be performed due to its location.

Table 5. Piezometric levels under conditions of maximum water levels in reservoir.

	Nov'77	Apr'78	Oct'09
Measuring point	(l/sec)		
Gallery 522	60.6	–	73.8
Bailey bridge spring	13.04	–	12.2
Kaludjerovo Vrelo spring	742	–	272
The Piva riverbed – I	–	–	9
The Piva riverbed –II*	242	146	42
The Piva riverbed – III**	505	396	195
The Piva riverbed – IV***	1070	1280	586
Reservoir	631–634	660.74	664.8

* 100 m downstream from Bailey bridge.
** 500 m downstream from Bailey bridge.
*** 150 m upstream from Kaludjerovo Vrelo spring.

Differences which occur in Piva river discharges are first of all a result of changed natural inflows from hinterland, during different hydrometeorological conditions. Besides that, a certain quantity of water flows through river sediment, without a possibility to be measured. The lower flow is, the higher error is, because percentual share of flow through the sediment is higher. Influence of sediment is the biggest at the most upstream section II and it decreases in downstream direction. Kaludjerovo Vrelo spring, which is not under influence of the reservoir, has different yields, dictated by recharging conditions, whereas the inflow from the drainage gallery and from the source which is under direct impact of the reservoir showed no significant changes.

In the period 1981–1982 the access tunnel used for the excavation of pile galleries was sealed and grouted as it had been determined that it transferred water from the reservoir to the rock mass downstream from the curtain and further to the Piva river. These works and repair grouting resulted, as established, in decreased water levels in the control piezometers and it is therefore a logical conclusion that they also resulted in reduced seepage from the reservoir through that zone. It means that water losses are probably smaller than the ones measured in the period 1977–1978. This was also confirmed by the measurements performed during 2009.

7 MEASUREMENTS AND TESTS PERFORMED DURING 2009

Additional measurements and investigations were performed during 2009 with the objective to provide additional information for making a decision related to necessity and scope of possible rehabilitation measures and grout curtain reparation. For such purpose the following tests were performed: measurement of the Piva river discharges, chemical analyses of water from drainage gallery and tracing test for ground water in both dam abutments.

For the purpose of establishing losses from reservoir and possible changes in discharges (by comparison with previous measurements), within

implementation of the envisaged tests, a series of measurements was performed at four measuring sections. Results of these measurements are presented in Chapter 1.5. In order to establish chemical composition of water which appears downstream from the grout curtain, and for which we assume that also partially passes through the grout curtain, the relevant chemical analyses were performed, and special attention was devoted to possible taking the grouting mass out.

Within the planned tests, testing by the tracing method was also performed. Each of the dam abutments was tested separately, by introducing the tracer into the upstream piezometers and monitoring occurrence of the tracer at selected downstream piezometers in left and right dam abutments, at Bailey bridge spring and in the drainage system. Tracing of ground water was performed by means of sodium-fluorescein ($C_2OH_{10}O_5Na_2$).

8 CONCLUSIONS

8.1 *Left abutment*

Previous multiyear analyses have shown that the curtain has reduced water level increment downstream from the curtain. Under the impact of the reservoir the HPP's operation, it increased by only 16 to 25% of the water column height in the reservoir above the minim water level and by 10 to 15% above the maximum water level. It means that the curtain fulfills it function of preventing water losses downstream, although in some zone this function has not resulted in the designed values. The reason for that is poor water-permeability of the massif, which had effect on the groutability of the rock mass.

Ground water levels registered in the observed piezometers show that the first half of grout curtain in the left abutment (up to P-2 piezometer) does not have completely satisfactory antifiltration effects, i.e. that there is an increased flow through the grout curtain in this zone. Testing performed by tracing of ground water confirms this conclusion, at which the zone between P-46 and P-49 piezometers, in the central part of the gallery, has been separated as the most delicate zone, with partial degradation of grout curtain. Passing of water around the grout curtain has also been registered.

However, it should be kept in mind that in the previous curtain functioning, which was analyzed in the course of preparation of annual reports and analyses of piezometer observations, deterioration of the current piezometric picture i.e. visible subsequent degradation of the grout curtain was not registered.

Even besides this fact and bearing in mind the results of performed testing, it is necessary to consider possibility of improvement—rehabilitation of the grout curtain in the zones which were identified as permeable zones, i.e. in the first part of the grout curtain and, first of all, in the zone between P-45 and P-48 piezometers. Otherwise, this process might in time bring to serious damages of the grout curtain.

8.2 *Right abutment*

Groundwater level downstream from the curtain in the right abutment is above the natural maximum level, as in the case of the left abutment. Results of regular observations and analyses made within this Study show that groundwater partly flows through and around the curtain and that its impact varies along the curtain. Testing performed by tracing of ground water confirms this conclusion. It was registered during trace testing that the tracer first appeared in the zone from P-11 to P-13 piezometers, i.e. in the zone where the three-row grout curtain ends. It has also been registered that the water passes around the grout curtain and appears at P-15 piezometer.

Having in mind a multiyear monitoring of ground water in the zone of right dam abutment, as well as the tracing test results, it can be said that the impact of the access tunnel intended for excavations (which passed through the grout curtain and has an influence on high ground water levels that appear in the zone at the beginning of the grouting gallery—in piezometers from Pk-3 to Pk-5), is still noticeable.

An intensive effect of draining structures has also been registered in the same zone. Also, monitoring of ground water levels and the tracing test indicate to weakening of the zone where the three-row grout curtain transits into the single-row grout curtain, as well as to the increased flow rates, in relation to tracing tests performed in 1978. Presence of the increased silicon content was established through chemical analyses of drainage water which passes through the grout curtain.

Although the results of ground water monitoring, as well as measurements of flows from the drainage gallery and from the registered source, do not indicate to increased losses from the reservoir, it is necessary to undertake activities related to rehabilitation and improvement of the grout curtain. This refers in the first place to the zone between PD-12 and P-13 piezometers. Special attention should be devoted to grouting methodology, with regard to the confirmed poor groutability of the rock mass.

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Numerical investigation of energy dissipation over various stepped Spillway

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ABSTRACT: The stepped Spillway is the most powerful hydraulic structure to dissipate large flow energy downstream from steep hills. This paper presents the flow characteristics for energy dissipation in various types of stepped Spillways: horizontal steps, steps with end sills, and inclined steps by numerical simulation. Adopting the Reynolds stress turbulence model, the mixture flow model for air-water two-phase flow was used to simulate the flow field over stepped Spillways. The mixture model is suitable method than VOF method to produce the interaction between entrained air bubbles and cavity recirculation in the skimming flow regime. The relationships between relative critical flow depth and relative energy loss on various stepped Spillways are presented and discussed. The steps with end sills have the most efficient configuration in terms of relative energy dissipation. For a certain discharge with a constant relative critical flow depth, the relative energy loss increases with increasing the number of steps. In addition, new correlation for the relative energy loss on horizontal steps is proposed for practical applications.

1 INTRODUCTION

When water flows over a stepped Spillway, the roughness elements or steps significantly increase the dissipation energy rate, and reduce the size of downstream energy dissipater and its cost of construction. A number of studies on flow in horizontal stepped Spillways and spillways were found in the papers and books published by Chanson (2002). With due consideration of energy dissipation, the characteristics of relative energy loss of flow can be compared to the drop number q^2/gH_T^3, where H_T = total drop height, g = gravitational acceleration, and q = flow discharge per channel width. The drop number strongly influences the relative energy loss. When the drop number decreases, the relative energy loss ratio increases (Peyras et al., 1992). In addition, the relative energy loss depends on the Spillway slope, that is, it is higher for mild slopes. Chaturabul (2002) investigated through laboratory studies, the energy loss and outlet velocity on stepped channels with end sills. For various stepped channel slopes, the heights of end sill were 5, 10, and 15 mm, respectively. The relationship between drop number and relative energy loss was presented. He inferred that the presence of an end sill increased the relative energy loss by about 8%.

Chinnarasri and Wongwises (2004) presented flow regimes and energy loss on Spillways with upward inclined steps. They concluded that the slope of the inclined steps decreased the outlet velocity and increased the relative energy loss by nearly 10%.

To find the best energy dissipation efficiency for flow on stepped Spillways, the objective of this study is to compare the relative energy dissipation among horizontal steps, inclined steps, and steps with end sills. The effects of number of steps and step geometry and the relationship between relative energy loss and relative critical flow depth are presented and discussed. The coefficient and exponent from regression analysis are proposed and compared.

Today, with the use of high-performance computers and more efficient Computational Fluid Dynamics (CFD) codes, the behavior of flow structures can be investigated numerically in reasonable time and with reasonable expense. In this study Energy Dissipation over Various Stepped Spillways was investigated by numerical method, therefore the finite volume computational fluid dynamics code was used to simulate the main characteristics of the flow over the stepped spillway. The mixture flow model is powerful tool than VOF model to simulation of two-phase flow because mixture model produce the interaction between entrained air bubbles and cavity recirculation in the skimming flow

regime, therefore mixture model was adapted in this study. The control-volume-based technique was used to solve the equations. The computation domain was divided into discrete control volumes by the unstructured mesh which has a high flexibility to fit the complex geometry and boundary of stepped spillway. The convective fluxes in the mean volume fraction, momentum and turbulence closure equations were discretized by employing a conservative, second-order accurate upwind scheme. The pressure-velocity coupling algorithm is the Pressure-Implicit with Splitting of Operators (PISO), which is based on the higher degree of the approximate relation between the corrections for pressure and velocity and may also be useful for transient calculations on highly skewed meshes.

2 NUMERICAL SETUP AND PROCEDURE

Adopting the Reynolds Stress turbulence Model (RSM), the mixture flow model for air-water two-phase flow was used to simulate the flow field over stepped Spillways. The mixture model differs from the Volume Of Fluid (VOF) model in three respects: (i) the mixture model allows the phases to be interpenetrating. The volume fractions α_k and α_q for a control volume can therefore be equal to any value between 0 and 1, depending on the space occupied by phase k and phase q. (ii) The mixture model allows the phases to move at different velocities, using the concept of slip velocities. (iii) There is interaction of the inter-phase mass, momentum and energy transfer in the mixture model. However, the VOF method does not compute the dynamics in the void or air regions. The complicated air-water flow will be formed under the strong action of the steps' disturbing nappe over spillways and the majority of the air will be trapped into water. Currently, the mixture model is very popular to simulate the characteristics of air-water two-phase flow. Two-phase flows are taken as continuous medium and coupled by the means of their interactions in the mixture model because the mixture model allows for the modeling of multiple, separate, yet interacting phases. (based on work presented by Cheng et al., 2006).

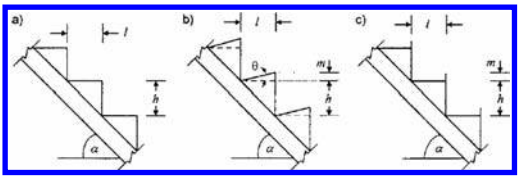

Figure 1. Configurations and notations of step, a) horizontal step, b) inclined step, c) step with end sill.

Three types of step were analyzed in this study. The dimensions of the step can be defined as h/l, where h = step height and l = horizontal length. Configurations and notations of step used in the present study are shown in Fig. 3. Details of inclined step and step with end sill dimensions are shown in Table 1.

Schematic of numerical model and boundary condition of simulation domain is shown in Fig. 2. The stepped Spillway models having widths of 0.40 m. The slopes of the stepped channel (α) are 30, 45, and 60° with the total drop heights of the stepped Spillway (H_T) 1.50, 2.12, and 2.60 m, respectively. All the air boundaries are defined as pressure boundaries with zero pressure. The inlet water boundary defined as a velocity inlet. The outlet boundary is also defined as a pressure boundary, so that the water can flow out freely.

Table 1. Details of inclined step and step with end sill dimensions.

SPILLWAY	α (degrees)	l (cm)	h (cm)	θ (degrees)	m (cm)
INCLINED STEP	30	13.0	7.5	0	0.00
				10	2.29
				20	4.73
				30	7.51
	45	10.6	10.6	0	0.00
				10	1.87
				20	3.86
				30	6.12
	60	7.5	13.0	0	0.00
				10	1.32
				20	2.73
				30	4.33
STEP WITH END SILL	30	13.0	7.5	0	5
				0	5
				0	5
	45	10.6	10.6	0	10
				0	10
				0	10
	60	7.5	13.0	0	15
				0	15
				0	15

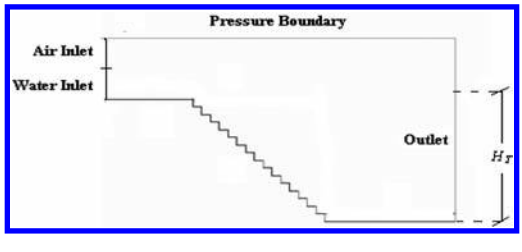

Figure 2. Schematic of numerical model and boundary condition of simulation domain.

3 SIMULATION RESULTS AND ANALYSIS

To find a suitable model for simulation of two-phase flow over stepped spillway, VOF, Mixture models and various turbulence models investigated by the Rezaei et al., 2009. Results showed that mixture flow model and Reynolds stress turbulence model (RSM), provide better results based on comparison with some experimental stepped Spillway models of Chinarsi and Wongwisess (2006) simulated. For example, Fig. 3 shows comparison of VOF and mixture models with experimental data of Chinarsi and Wongwisess (2006). In this graph, E_L is energy loss of flow. Experimental models of Chinarsi and Wongwisess (2006) are shown in Table 2.

To numerically investigate the effect of step geometry on the relative energy loss, the geometries of step are as follows: For the inclined step, the angles of inclined step, θ, were 10, 20, and 30°, respectively. For the steps with end sills, the heights of end sill were 5, 10, and 15 mm, respectively. For both cases, the edge of the step is estimated above the horizontal level. This incremental height can be named as the characteristic height (m). The characteristic heights of the step inclination For both cases, are shown in Table 1.

Details of numerical data for horizontal and inclined steps and steps with end sills for Spillway slopes of 30°, 45°, and 60°, H_T = 2.12 m, and h = 0.106 m are shown in Tables 2, 3, 4 and 5, respectively. Boes and Hager (2003) presented that

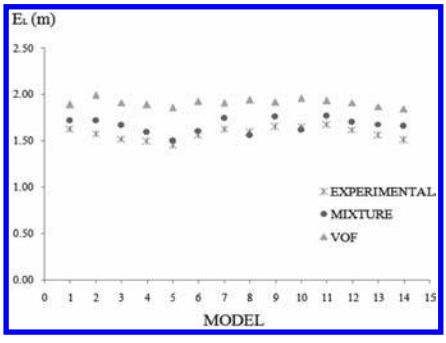

Figure 3. Comparison of VOF and mixture models with experimental data of Chinarsi and Wongwisess (2006).

Table 2. Experimental models of Chinarsi and Wongwisess (2006).

	MODELS													
PARAMETERS	I	II	III	IV	V	VI	VII	VIII	IX	X	XI	XII	XIII	XIV
θ	0	0	0	0	0	10	10	20	20	30	30	0	0	0
m	-	-	-	-	-	-	-	-	-	-	-	0.05	0.05	0.05
V_T(m/s)	3.43	3.62	3.80	3.90	4.05	3.69	3.47	3.60	3.41	3.52	3.36	3.44	3.66	3.84
flow type	sk	sk	sk	sk	sk	sk	sk	sk	sk	sk	sk	sk	sk	sk
dc/h	0.89	1.02	1.12	1.22	1.35	1.13	1.01	1.13	1.01	1.13	1.01	0.99	1.15	1.28
E_L(m)	1.63	1.58	1.52	1.49	1.45	1.56	1.63	1.59	1.65	1.64	1.68	1.62	1.56	1.51

Note: SK = skimming flow regime.

Table 3. Details of numerical data for horizontal and inclined steps for spillway slope = 30°, flow regime is skimming.

θ	dc/h	VT(m/s)	C	h_{90}	E_L (m)
0	1.44	2.91	0.690	0.089	1.20
0	1.58	3.08	0.669	0.100	1.16
0	1.72	3.29	0.652	0.109	1.10
0	1.90	3.42	0.629	0.115	1.07
0	2.05	3.59	0.614	0.123	1.03
0	2.20	3.71	0.587	0.129	0.99
10	1.44	2.98	0.711	0.120	1.17
10	1.58	3.22	0.701	0.126	1.11
10	1.72	3.40	0.677	0.134	1.06
10	1.90	3.54	0.655	0.141	1.03
10	2.05	3.67	0.643	0.145	0.99
10	2.20	3.80	0.638	0.148	0.96
20	1.44	2.70	0.722	0.138	1.25
20	1.58	3.20	0.699	0.143	1.11
20	1.72	3.31	0.691	0.150	1.09
20	1.90	3.46	0.661	0.153	1.05
20	2.05	3.59	0.645	0.156	1.02
20	2.20	3.75	0.620	0.158	0.97
30	1.44	2.70	0.724	0.156	1.25
30	1.58	3.10	0.707	0.161	1.14
30	1.72	3.31	0.691	0.167	1.08
30	1.90	3.42	0.668	0.170	1.06
30	2.05	3.57	0.655	0.174	1.02
30	2.20	3.74	0.625	0.175	0.97

Table 4. Details of numerical data for horizontal and inclined steps for spillway slope = 45°, flow regime is skimming.

θ	dc/h	VT(m/s)	C	h_{90}	E_L (m)
0	1.02	3.23	0.721	0.093	1.72
0	1.12	3.65	0.690	0.106	1.58
0	1.22	3.76	0.676	0.115	1.56
0	1.35	3.90	0.651	0.122	1.52
0	1.45	4.04	0.634	0.129	1.47
0	1.55	4.17	0.606	0.136	1.43
10	1.01	3.10	0.731	0.126	1.74
10	1.13	3.58	0.721	0.132	1.61
10	1.25	3.79	0.701	0.143	1.54
10	1.31	3.93	0.679	0.149	1.50
10	1.45	4.07	0.667	0.152	1.47
10	1.55	4.14	0.661	0.155	1.45
20	1.01	3.05	0.748	0.145	1.76
20	1.13	3.51	0.730	0.151	1.64
20	1.26	3.68	0.713	0.158	1.59
20	1.35	3.84	0.685	0.161	1.54
20	1.45	3.97	0.671	0.163	1.50
20	1.55	4.01	0.641	0.167	1.49
30	1.01	3.03	0.750	0.166	1.77
30	1.13	3.41	0.735	0.170	1.66
30	1.23	3.67	0.719	0.176	1.60
30	1.32	3.80	0.692	0.181	1.56
30	1.45	3.94	0.676	0.183	1.53
30	1.55	3.96	0.646	0.184	1.51

$h_w = h_{90}(1\text{-}C)$ is an analytically computed value in two-phase flow with the mixture depth h_{90} = h(C = 0.90) and the depth-averaged air concentration C. h_W = water depth; h_{90} = h(C = 0.90) i.e., a flow depth

Table 5. Details of numerical data for horizontal and inclined steps for spillway slope = 60°, flow regime is skimming.

θ	dc/h	VT(m/s)	C	h90	EL (m)
0	0.83	3.49	0.751	0.097	2.12
0	0.91	3.70	0.723	0.111	2.05
0	0.99	3.94	0.698	0.119	1.97
0	1.10	4.10	0.680	0.128	1.92
0	1.20	4.23	0.665	0.132	1.88
0	1.30	4.36	0.641	0.143	1.83
10	0.83	3.36	0.761	0.132	2.15
10	0.91	2.66	0.753	0.138	2.38
10	0.99	4.10	0.736	0.150	1.90
10	1.10	4.25	0.707	0.156	1.85
10	1.20	4.39	0.694	0.161	1.80
10	1.30	4.53	0.678	0.164	1.75
20	0.83	3.24	0.773	0.152	2.19
20	0.91	3.84	0.766	0.158	1.99
20	0.99	3.97	0.741	0.165	1.95
20	1.10	4.15	0.713	0.169	1.89
20	1.20	4.27	0.697	0.171	1.85
20	1.30	4.41	0.675	0.177	1.80
30	0.83	3.25	0.782	0.174	2.19
30	0.91	3.73	0.769	0.178	2.03
30	0.99	3.97	0.755	0.184	1.94
30	1.10	4.12	0.723	0.190	1.90
30	1.20	4.29	0.701	0.195	1.84
30	1.30	4.44	0.672	0.198	1.78

with a local air concentration of 90%; V_T = Total depth-averaged mixture velocity taken from the velocity profiles at downstream of Spillway.

4 DIMENSIONAL ANALYSIS

Based on work presented by Chinarsi and Wongwisess (2006), the first step is to develop a correlation to select parameters that have an influence on the energy loss of flow. These parameters are focused on the input parameters to keep the correlation simple. The energy loss of flow on a stepped Spillway, E_L, depends on the total discharge per unit width of Spillway, q, the total flow head E_0, the step height h, the step length l, the characteristic height of the step m, and the gravitational acceleration g. For the inclined step, the characteristic height is the increment of step above the horizontal level. For the step with end sill, the characteristic height is the height of the end sill. These variables can be functionally expressed as

$$E_L = E_L(q, E_0, h, l, m, g) \quad (1)$$

As the critical flow depth d_c in a rectangular channel can be expressed as $\sqrt[3]{(q^2/g)}$, using the Buckingham Pi theorem, the variables in Eq. (1) can be expressed in non-dimensional form as

Table 6. Details of numerical data for steps with end sills for various spillway slopes, flow regime is skimming.

α	dc/h	m	VT(m/s)	C	h90	EL (m)
30	1.44	0.05	2.88	0.73	0.09	1.22
30	1.58	0.05	3.01	0.7	0.105	1.18
30	1.72	0.05	3.17	0.68	0.115	1.14
30	1.90	0.05	3.33	0.66	0.121	1.11
30	1.44	0.1	2.77	0.74	0.095	1.25
30	1.58	0.1	2.91	0.722	0.108	1.22
30	1.72	0.1	3.06	0.701	0.119	1.18
30	1.90	0.1	3.21	0.679	0.123	1.15
30	1.44	0.15	2.71	0.759	0.099	1.26
30	1.58	0.15	2.85	0.735	0.112	1.23
30	1.72	0.15	2.97	0.717	0.122	1.21
30	1.90	0.15	3.1	0.69	0.129	1.18
45	1.02	0.05	3.16	0.761	0.097	1.75
45	1.12	0.05	3.31	0.724	0.111	1.71
45	1.22	0.05	3.49	0.711	0.121	1.66
45	1.35	0.05	3.65	0.681	0.128	1.61
45	1.02	0.1	3.05	0.778	0.101	1.79
45	1.12	0.1	3.2	0.745	0.114	1.75
45	1.22	0.1	3.37	0.729	0.126	1.70
45	1.35	0.1	3.51	0.702	0.131	1.67
45	1.02	0.15	3.02	0.865	0.111	1.80
45	1.12	0.15	3.14	0.828	0.128	1.77
45	1.22	0.15	3.27	0.813	0.138	1.74
45	1.35	0.15	3.39	0.783	0.145	1.72
60	0.83	0.05	3.48	0.78	0.102	2.12
60	0.91	0.05	3.65	0.759	0.116	2.07
60	0.99	0.05	3.84	0.735	0.127	2.01
60	1.10	0.05	4.01	0.714	0.134	1.96
60	0.83	0.1	3.35	0.811	0.811	2.04
60	0.91	0.1	3.52	0.781	0.78	1.98
60	0.99	0.1	3.71	0.753	0.754	1.91
60	1.10	0.1	3.87	0.734	0.734	1.86
60	0.83	0.15	3.33	0.841	0.108	2.18
60	0.91	0.15	3.46	0.811	0.124	2.14
60	0.99	0.15	3.6	0.782	0.133	2.10
60	1.10	0.15	3.73	0.765	0.143	2.07

$$\frac{E_L}{E_0} = \frac{E_L}{E_0}\left(\frac{d_C}{h}, \frac{h}{l}, \frac{m}{h}, N\right) \quad (2)$$

The dimensionless term E_L/E_0 = relative energy loss; dc/h = relative critical flow depth; h/l = Spillway slope α, that is, tan α = h/l; m/h = relative step height; and N = number of steps in Spillway. The terms dc/h, m/h, and N are investigated in detail in this study.

5 FLOW CHARACTERISTICS ON STEPPED SPILLWAYS

Based on work presented by Chinarsi and Wongwisess (2006), there are three types of flow regimes on stepped Spillways, that is, skimming flow, transition flow, and nappe flow. Generally, for large discharge, the free surface of the flow is smooth and air entrainment is small. Flow depth is small compared to the step height. For intermediate discharge, the free jet disappears. The free surface of

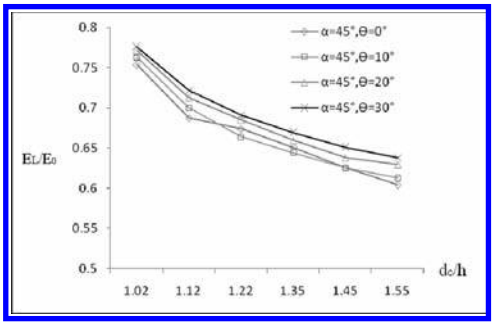

Figure 4. Relative energy loss and relative critical flow depth for inclined steps.

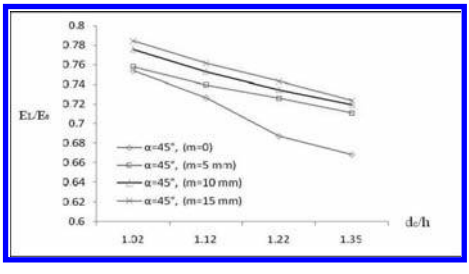

Figure 5. Relative energy loss and relative critical flow depth for steps with end sills.

the flow is wavy with spray. For a smaller discharge, a free-falling nappe is found at the brink of the step while a hydraulic jump is observed on the step face.

The relation between relative energy loss (E_L/E_0) and relative critical flow depth (d_c /h) for 45° Spillways of inclined steps and steps with end sills are shown in Figs. 4 and 5. Generally, the relative energy loss increases as the relative critical flow depth decreases. In skimming flow, the effect of characteristic height is clearly observed. For inclined steps or steps with end sills, the presence of the characteristic height has the effect of increasing the relative energy loss. For example, at $d_c/h = 1.35$, the relative energy loss for the horizontal step is 0.647. For an inclined step (Fig. 4), the relative energy losses are 0.643, 0.659, and 0.674 for characteristic heights of 19, 39, and 61 mm, respectively. For a step with end sill (Fig. 5), the relative energy losses are 0.644, 0.663, and 0.677 for characteristic heights of 5, 10, and 15 mm, respectively. It is clearly seen that the step with end sill requires a lower characteristic height than the inclined step to produce the same relative energy loss.

6 EFFECT OF NUMBER OF STEPS

The energy loss of flow over a horizontal step can be computed by equations proposed by

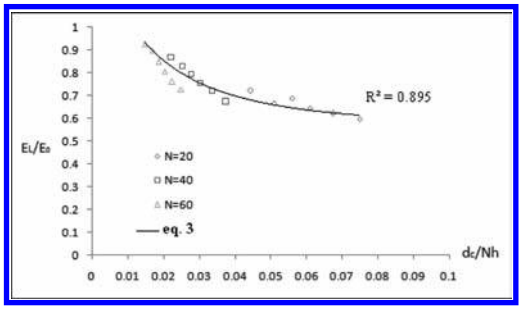

Figure 6. Effect of number of steps on energy dissipation.

Rajaratnam (1990) and Chanson (2002). Although these existing equations are based on assumption of the establishment of uniform skimming flow. This assumption would imply a high stepped Spillway with many steps, which may not be typical of stepped structures. To evaluate the effect of the number of steps, the relative energy loss on horizontal steps, E_L/E_0, the 45° Spillway slope at steps 20, 40, and 60 was analyzed. Fig. 6 is presented the relationship between E_L/E_0 and d_c/Nh. Generally, for a certain discharge with a constant relative critical flow depth dc/h, the relative energy loss increases with increase the number of step. It should be noted that the total Spillway height, H_T, is the product of step height and number of step (Nh). The correlation for the relative energy loss on horizontal steps is obtained by fitting the numerical data as shown by the dashed line in Fig. 6 and defined as

$$\frac{E_L}{E_0} = 1.473\exp\left(-159.3\,\frac{d_c}{Nh}\right) + 0.845\exp\left(-4.43\,\frac{d_c}{Nh}\right) \quad (3)$$

The correlation can be used in design practice for preliminary evaluates of the energy loss.

7 CONCLUSIONS

Mixture flow model for air-water two-phase flow and Reynolds Stress turbulence Model (RSM), are suitable models for simulation of Two-phase flow on various stepped spillway.

In skimming flow, because the characteristic height of the end sill increases the volume of trapped water, the circulating vortex was greatest compared to other stepped Spillways. The step geometry influences the relative energy loss. As the characteristic height of the step on various stepped spillway increases, the relative energy loss increases. To maintain the same relative energy

loss, the characteristic height of the inclined step is higher than that of the end sill. The relationship between relative critical flow depth and relative energy loss can be applied for practical applications. In addition, for a constant step height with a certain discharge the relative energy loss increases when the number of steps increases.

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Diversas actuaciones realizadas en las presas que regulan el río Gállego

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RESUMEN: Durante los últimos años se han realizado diversas obras importantes en varias presas que regulan la cuenca del río Gállego, que se describen en este artículo. En Lanuza se ha proyectado la construcción de un ascensor inclinado a la intemperie para facilitar el acceso a las galerías de la presa y realizar actividades de mantenimiento e inspección durante las épocas de nevadas. En La Peña se ha sustituido una compuerta automática del aliviadero por dos compuertas vagón para mejorar la explotación del embalse y la regulación del río Gállego. En Ardisa se ha realizado una obra de gran reparación que afecta a los desagües de fondo y al aliviadero y se está proyectando una obra para ensanchar la coronación. En La Sotonera se realizó una obra de gran reparación para dotarla de desagüe de fondo y de aliviadero. Está en fase de proyecto una obra con diversas actuaciones sobre compuertas, espaldones y coronación.

ABSTRACT: In recent years there have been several important works in some dams wich regulate the Gállego river basin, as described in this article. It's planned the construction of an inclined elevator in the open air in Lanuza dam to facilitate access to the galleries of the dam and carry out maintenance and inspection activities during periods of snowfall. At La Peña dam, an automatic floodgate has been replaced with two fixed wheel gates to improve the operation of the reservoir and regulation of river Gállego. A major repairs work has been done in Ardisa dam, affecting the bottom outlets and spillway and a project to widen the coronation is being drafted. A major repairs work was carried out to equip the dam with bottom outlet and spillway in La Sotonera dam. Another project is being drawn up to design several actions on gates, shoulders and crest.

1 INTRODUCCIÓN

El río Gállego discurre principalmente en dirección norte-sur desde el Pirineo Central (valle de Tena) hasta su desembocadura en el río Ebro en Zaragoza. Su estrecha y alargada cuenca vertiente es de 3.969 km^2 y se encuentra íntegramente en Aragón. El río tiene numerosos afluentes entre los que destacan los ríos Aurín y Asabón en la margen derecha, y los ríos Caldarés, Guarga y Sotón en su margen izquierda.

El río Gállego ha sido regulado en cabecera para el aprovechamiento hidroeléctrico de sus aguas y en su tramo medio existen embalses y canales para uso agrícola principalmente.

Además de los 13 pequeños embalses hidroeléctricos en el Alto Gállego, los embalses más importantes son Lanuza, Búbal (ambos de usos hidroeléctrico y agrícola), La Peña (el más antiguo), Ardisa y Sotonera (éste el más grande de la cuenca y ambos para uso principalmente agrícola).

Las aguas de los ríos Gállego y Cinca proporcionan agua para los regadíos del Alto Aragón. Este sistema de regadíos está integrado por las zonas regables de Monegros I, Canal de Cinca y Monegros II.

En el embalse de Ardisa existe un trasvase de aguas de este embalse hacia el de La Sotonera (Canal del Gállego). En la Sotonera comienza el canal de Monegros, cuyo tramo I tiene una capacidad máxima de 90 m^3/s. Del canal de Monegros parte el canal de la Violada, que abastece a los regadíos de los Llanos de la Violada y que tiene una capacidad máxima de 6 m^3/s.

2 DESCRIPCIÓN DE LAS PRESAS

Las presas objeto de este artículo son Lanuza, La Peña, Ardisa y La Sotonera, y las describimos a continuación.

2.1 *Presa de Lanuza*

La presa de Lanuza se sitúa sobre el río Gállego, al norte de la provincia de Huesca y muy próxima a la frontera francesa, a unos 500 m aguas abajo del pueblo que le dio nombre. El objetivo básico de la construcción de la presa de Lanuza fue la regulación de la cabecera del río Gállego, para su aprovechamiento en el Plan de Riegos del Alto Aragón, así como para la producción de energía hidroeléctrica en la central situada en Pueyo de Jaca aguas abajo de la presa.

La presa de Lanuza es una presa bóveda de hormigón, de doble curvatura, cuya directriz horizontal tiene tres centros distintos. La altura máxima es de 79,50 metros sobre cimientos y de 68,50 metros sobre cauce. La coronación tiene una longitud de 176,33 metros y su anchura estructural era de 3,00 metros, pero fue ampliada con una ménsula a cada lado hasta alcanzar los 6,50 metros.

La cota de coronación es la 1.286,50, situándose el NMN a la cota 1.275,50. La presa dispone de una red interior de galerías accesible desde el paramento aguas abajo a través de unas bocas o salidas al exterior situadas en ambas márgenes a diferentes cotas. Estas salidas también facilitan el acceso a la galería perimetral interior, que es la única manera de llegar de una margen a la otra por el interior del cuerpo de presa. Tanto los accesos exteriores como la galería perimetral interior están formados por tramos de escaleras con mucha pendiente y en un estado de conservación que hace que su utilización sea poco segura, sobre todo en época invernal.

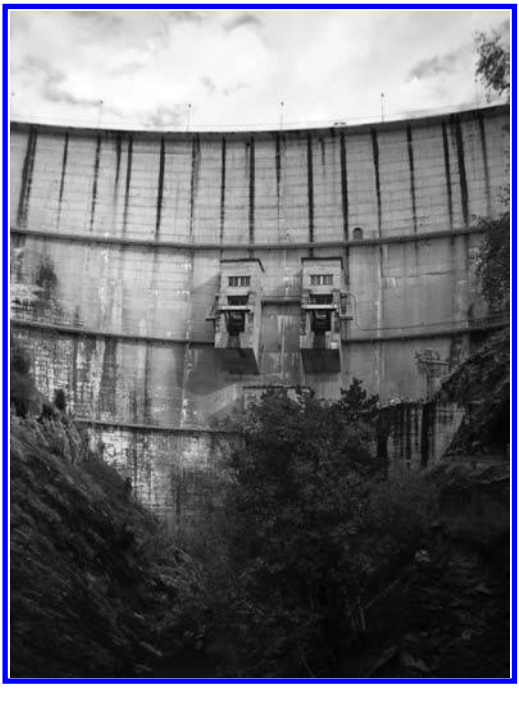

Figura 1. Vista de la presa de Lanuza.

2.2 *Presa de La Peña*

La presa de La Peña es la más antigua del río Gállego. Es una presa de tipo arco-gravedad construida con sillares de piedra caliza fijados con mortero. Fue terminada en 1913.

Se sitúa en la confluencia del río Asabón y el río Gállego, y tiene 59 metros de altura y 112 metros de longitud de coronación. Está dotada de cuatro tomas de agua con sendas válvulas de compuerta de 0,60 metros de diámetro situadas de forma que se puede vaciar casi todo el embalse, a excepción de unos 1,5 hm^3 que lo hacían por los desagües de fondo.

También consta de un aliviadero formado por diez túneles con una capacidad de 2.900 m^3/s, cada uno de los cuales viene precedido de una compuerta automática. Estas compuertas tienen varios problemas, siendo el más importante de ellos que cuando el nivel de agua sube hasta hacer funcionar las compuertas, estas no vuelven a cerrarse hasta que el nivel del agua ha bajado casi 3 metros, por lo que cuando se accionan las compuertas automáticamente se pierde una capacidad de embalse importante además de carecer su control para regular.

2.3 *Presa de Ardisa*

La presa de Ardisa está situada sobre la línea divisoria de las provincias de Zaragoza –estribo derecho– y Huesca –estribo izquierdo–. Está en explotación desde el año 1925.

El objeto de la construcción de la presa es derivar aguas del río Gállego al embalse de La Sotonera a

Figura 2. Vista de la presa de La Peña.

Figura 3. Vista de la presa de Ardisa.

Figura 4. Vista de la presa de La Sotonera.

través del canal del Gállego, contribuyendo a su alimentación. Por lo tanto, el destino prioritario de sus aguas es el riego, aunque también se le da un uso secundario hidroeléctrico.

La presa de Ardisa es una presa de gravedad, de sección trapecial de hormigón ciclópeo y planta curva, con el paramento aguas abajo escalonado y una altura máxima de 34,72 m desde coronación y 29,54 m desde cauce. El ancho de coronación es de 3,10 m útiles.

Actualmente cuenta con un aliviadero de labio fijo, y una capacidad de embalse de 1,871 Hm^3. Consta de una toma para el canal del Gállego formada por 8 conductos, un desagüe intermedio bajo la toma del canal formado por dos túneles y un desagüe de fondo consistente en cuatro conductos con sus correspondientes compuertas.

2.4 *Presa de La Sotonera*

La presa de La Sotonera está situada en los ríos Sotón y Astón, en el término municipal de Alcalá de Gurrea (Huesca).

El río Sotón es afluente por la margen izquierda del río Gállego que, a su vez, lo es del río Ebro. De la misma forma, el río Astón es afluente del río Sotón por la margen derecha, recibiendo por su izquierda la vaguada del Vacas y por su derecha la de la Alberca de Alcalá.

En realidad se trata de tres presas cuyas coronaciones se enlazan sin solución de continuidad que, enumeradas de izquierda a derecha en el sentido del río, se denominan respectivamente: Sotón, Astón y Vacas, y Alberca de Alcalá.

La presa de La Sotonera es una presa de materiales sueltos, homogénea y con espaldones permeables de escollera.

La longitud total de coronación es de 3.858,80 m y la altura máxima sobre cimientos de 34 m, que corresponde al tramo Astón y Vacas en su cruce con el valle de Vacas. La coronación tiene una anchura total de 6,00 m incluidas las aceras y el pretil, teniendo una anchura útil de 4,30 m de firme de carretera.

El talud aguas arriba va protegido del oleaje mediante un recubrimiento de losas de 0,50 m de espesor, de arenisca o de hormigón según las zonas y la época de construcción.

El embalse se llena principalmente con agua proveniente del río Gállego a través del canal del Gállego desde la presa de Ardisa. Tiene una capacidad de 189,22 Hm^3, y su uso es principalmente agrícola, con usos secundarios para abastecimiento e hidroeléctrico.

3 OBJETO DE LAS ACTUACIONES

Las actuaciones que se han realizado o está previsto realizar en las presas de la cuenca del Gállego han tenido diferentes principios, estando todos relacionados con la funcionalidad, seguridad o mantenimiento y conservación de las presas.

4 DESCRIPCIÓN DE LAS ACTUACIONES

4.1 *Proyecto de adecuación al RTSPE de la presa de Lanuza*

Las obras destinadas a las mejoras en los accesos de la presa de Lanuza se dividen en tres grupos independientes muy definidos:

a. Mejoras en el acceso a las pasarelas exteriores del paramento aguas abajo de la presa, mediante un ascensor inclinado. Actualmente se realiza mediante una escalera exterior.
b. Mejoras en las condiciones e instalaciones de las pasarelas exteriores situadas en el paramento aguas abajo de la presa.
c. Mejoras en las galerías interiores que recorren el cuerpo de presa.

A continuación amplían estas tres actuaciones.

4.1.1 *Acceso a pasarelas exteriores*

La presa de Lanuza presenta unas galerías exteriores en el paramento aguas abajo con un acceso difícil desde una escalera exterior que recorre la totalidad del paramento.

Esta escalera presenta el problema de encontrarse con tramos que presentan dificultades tanto por pendiente de la escalera como por el tránsito peligroso provocado por la humedad existente en la propia escalera. Estas dificultades se incrementan en épocas de nevadas, llegando a imposibilitar su acceso.

Se plantea la instalación de un ascensor exterior inclinado sobre la ladera derecha que permita el fácil acceso a las pasarelas. Este ascensor evitará el problema del difícil acceso por las escaleras existentes. De esta solución se ha valorado muy positivamente la mejora en las condiciones de seguridad que se consiguen con la instalación del ascensor.

Esta instalación consta de varios elementos diferenciados. En primer lugar, se construirá el cuarto de máquinas con piezas prefabricadas y vallado perimetral. A lo largo de la trayectoria del ascensor se colocarán dados de hormigón que servirán para cimentar los apoyos de las vigas que sirven de raíl al ascensor. Se instalará una escalera metálica a lo largo de toda la trayectoria del ascensor que sirva de apoyo en caso de emergencias o de mantenimiento. En las cuatro paradas proyectadas se ejecutará una plataforma de acceso de hormigón armado, con barandilla perimetral y puerta de acceso secundaria en el lado de acceso al ascensor. Todo el conjunto va complementado con un sistema de evacuación de aguas que evitará el encharcamiento del foso, que se ejecutará también de hormigón armado.

El ascensor inclinado recorrerá una distancia aproximada de 83 m salvando una altura de aproximadamente 46 m. Su capacidad será de 8 personas y estará preparado para la intemperie.

Figura 5. Infografía que representa la situación del ascensor.

4.1.2 *Pasarelas exteriores situadas en el paramento aguas abajo de la presa*

Las galerías exteriores del paramento aguas abajo, formadas por piezas de trámex, presentan algún punto en mal estado. Además, en numerosos puntos se encuentran mal adaptados al paramento de la presa, dejando huecos que pueden resultar peligrosos para el tránsito de personas.

Por tanto, es necesaria la adecuación de estos tramos de pasarela mediante la colocación de tramex nuevos adaptados al paramento.

4.1.3 *Escaleras de galerías interiores en el cuerpo de presa*

Las escaleras de las galerías interiores de la presa presentan problemas debidos a las fuertes pendientes y a la constante humedad existente, además existen escalones prácticamente sin huella. La adecuación consistirá en la colocación de tramex, barandillas y puertas a lo largo de todas escaleras de las galerías.

4.2 *Sustitución de alza del aliviadero del embalse de La Peña*

Las compuertas automáticas del aliviadero del embalse de La Peña se construyeron en el año 1921, en una época en la que se generalizaron este tipo de cierres. Sin embargo, estas compuertas tienen el inconveniente del fallo relativamente frecuente de su automatismo, debido en gran parte a su funcionamiento ocasional. Además, no permiten su control, ya que cuando la cota del embalse llega a la 539,05 msnm se dispara la primera alza, sin permitir una correcta regulación de caudal, además de no tener control sobre ellas hasta que la cota desciende y la lámina de agua permite que las alzas se recuperen. Esto conlleva que pequeñas avenidas provoquen una pérdida de volumen de agua embalsada.

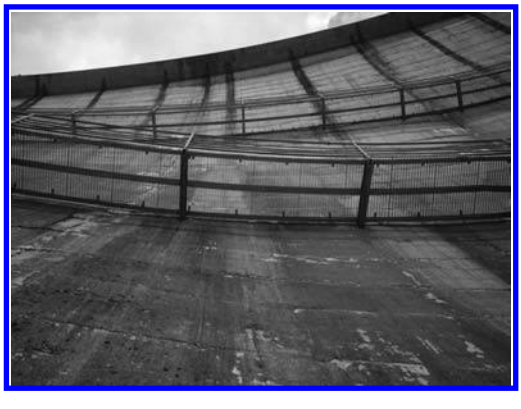

Figura 6. Vista de las pasarelas exteriores.

Figura 7. Vista de las escaleras interiores.

Figura 8. Estado final de las compuertas de La Peña.

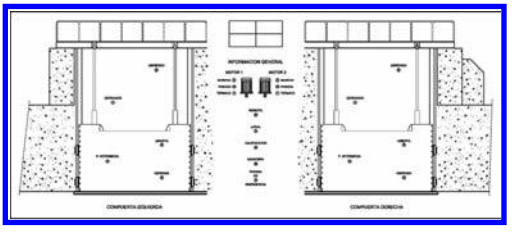

Figura 9. Esquema de las compuertas de La Peña.

Con la instalación de unas compuertas vagón, sustituyendo al alza nº1, se consigue regular con mayor precisión, manteniendo una cota en la cual no se dispararan el resto de las compuertas automáticamente y pudiendo mantener una capacidad mayor de embalse durante más tiempo. De esta forma, se mantendría un nivel de embalse preciso, consiguiendo abastecer a través de las tomas de riego el caudal óptimo durante más tiempo y evitando evacuar agua a través de las compuertas automáticas de aliviadero en caso de que no sea imprescindible.

La obra civil consistió en la construcción de una pila central que divide el vano en dos y una pilas laterales para el montaje de los hierros fijos de las compuertas. También se instaló una caseta para albergar el pupitre de mando y control de las compuertas y unas pasarelas de acceso que, situadas sobre los vanos incluyen las vigas soporte de los cilindros de accionamiento de las compuertas.

Las compuertas instaladas son de dimensiones 6 × 3,5 m. Tienen la particularidad de que sus cilindros de accionamiento oleohidráulico se recogen dentro de la misma compuerta. Las compuertas también disponen de vertedero.

4.3 *Proyecto de gran reparación de la presa de Ardisa*

Debido al estado en el que se encontraba la presa, se planteó la necesidad de acometer una serie de obras para adaptarla al nuevo RTSPE. Las obras que se definieron se detallan en los puntos que se relacionan a continuación.

4.3.1 *Aliviadero*

El aliviadero que existía antes de acometer las obras constaba de cuatro vanos de 18,50 m. de luz, cerrados por compuertas de sector con 6,50 m. de altura, de funcionamiento automático por flotación, situándose la cota máxima de las compuertas a la de coronación.

Después de analizar las diferentes alternativas, se decidió acometer la reconstrucción del antiguo aliviadero de labio fijo con la cresta del vertedero situada a la cota 420,00 msnm, junto con otras actuaciones que incluyen la consolidación de la cubeta que se crea mediante inyecciones, el desmontaje de las compuertas existentes, la demolición de la cresta

Figura 10. Vista del nuevo aliviadero.

del aliviadero existente hasta la cota 414,10 msnm y recrecido de los cajeros del canal de descarga. También se plantea la necesidad de consolidar el cuenco amortiguador a la salida del canal de descarga mediante escollera, así como el dragado del embalse en el lado de aguas arriba del nuevo labio fijo.

4.3.2 *Toma del canal del Gállego*

En el estribo izquierdo de la presa se sitúa la toma del Canal del Gállego, con 105 m^3/s de capacidad de diseño (limitado por el Canal a 90 m^3/s) que trasvasa las aguas del río Gállego al embalse de La Sotonera.

Las obras a realizar en la toma consisten en la puesta en funcionamiento de otras seis compuertas vagón de 2,80 × 5,50 m. similares a las existentes previamente y la demolición de los 40 cm. inferiores del muro situado aguas arriba de las compuertas, consiguiendo aumentar la capacidad de la toma aún cuando se baja el máximo nivel del embalse.

Para la instalación de las compuertas, se construyó dentro de cada uno de los tres vanos afectados una pila, que los divide en dos de 2,80 m de luz.

Las pilas de la toma se alargan hacia aguas abajo, para la colocación de unas ataguías, que junto con las existentes aguas arriba, permiten aislar cualquier compuerta, posibilitando su reparación manteniendo las otras en funcionamiento.

Para facilitar las operaciones con las compuertas se dispone un puente grúa en el edificio de la toma.

Durante la construcción de esta unidad se detectaron fugas en la salida de la toma del Canal, con lo que se realizó una campaña de inyecciones para eliminar esta circunstancia.

4.3.3 *Desagües de fondo*

Los desagües de fondo se encontraban constituidos por cuatro conductos rectangulares de 0,80 × 1,60 m., cerrados por compuertas de paramento aguas arriba. Al llevar años sin funcionar se había producido su aterramiento.

Para su puesta en servicio se procedió a la extracción del material sedimentado en sus proximidades. En cada uno de los conductos se limpió la superficie interior eliminándose los 5 cm. primeros. Posteriormente se colocó un conducto de acero de 1,50 × 0,70 metros de 8 mm de espesor con rigidizadores, inyectándose un mortero sin retracción entre su parte exterior y el hormigón de la presa.

Figura 11. Vista de las casetas de los desagües de fondo y remodelación del cuenco amortiguador.

Aguas abajo de la presa se transformó la sección del conducto a 0,90 × 1,20 m. por medio de una transición, agrupándose de dos en dos en sendas casetas de válvulas.

Cada una de estas casetas contiene los elementos necesarios para el manejo de las válvulas de los conductos y un puente grúa para facilitar las operaciones de reparación.

Cada conducto tiene, en su parte final, dos compuertas Bureau de 0,90 × 1,20 m.

Para acceder a las casetas de válvulas se construyó un acceso de 320 m de longitud con pendiente máxima del 12% y 3 m de anchura de calzada y cuneta.

Este acceso comunica la carretera de la margen izquierda, pasando por encima del desagüe de restitución de caudales al río Gállego con un puente, con una explanada de maniobra en el pie de aguas abajo de la presa.

Las dos casetas están unidas entre sí y con el acceso, mediante una explanada de hormigón de 4,20 m de anchura.

4.3.4 *Coronación de la presa*

Con objeto de compatibilizar la coronación de la presa con la altura de la lámina de agua provocada por la avenida de proyecto y la avenida extrema, y cumplir con los resguardos establecidos en las guía técnicas del Comité Nacional Español de Grandes Presas, es necesario aumentar el espacio libre bajo el puente del camino de coronación sobre el aliviadero, así como elevar la coronación de la presa.

La solución planteada es realizar un recrecido hasta la cota 423,40 msnm, es decir, teniendo en cuenta que la parte superior del parapeto actual resistente se encuentra a la cota 423,19 msnm resulta un recrecido con zuncho corrido de 21 cm de altura.

Además, se está ejecutando la sustitución de la estructura del puente del camino sobre el aliviadero, de manera que se consiga el hueco libre necesario para el paso del agua en caso de avenida. Aprovechando esta circunstancia, se plantea aumentar el ancho de la coronación en este tramo, de manera que admita dos carriles de circulación, dejando pendiente el resto de la coronación para otra actuación futura.

4.3.5 *Otras actuaciones*

Aprovechando la ejecución de esta obra, se realizan una serie de actuaciones complementarias consistentes en acondicionamientos o derribos de edificios, adaptación de instalaciones eléctricas y mejora de los caminos de acceso.

4.4 *Proyecto de adecuación de la presa de Ardisa al RTSPE*

La presa de Ardisa dispone de accesos a través de las dos márgenes. Al ejecutar el proyecto de Gran Reparación, se ensanchó el puente sobre el aliviadero para permitir dos carriles de tráfico, pero el resto de la coronación se mantuvo con la misma anchura.

Para conseguir un ancho de coronación continuo a lo largo de toda la coronación se plantea el proyecto de adecuación de la presa de Ardisa al RTSPE, con lo que se consigue cumplir el Reglamento en el sentido de mantener la seguridad y las buenas condiciones de los accesos desde ambas márgenes.

Está previsto ensanchar el tablero con un voladizo hacia aguas abajo a lo largo de toda la presa, conectando con el camino ya ensanchado en las obras del proyecto de Gran Reparación en la zona del puente sobre el aliviadero.

Junto con esta actuación, también se acometerán otras menores o complementarias.

4.5 *Proyecto de gran reparación de la presa de La sotonera*

La presa de La Sotonera tenía tres tomas, situadas a diferentes alturas, que derivaban agua al canal de Monegros. En fechas recientes, se utilizó la llamada "toma inferior" para instalar una central hidroeléctrica a pie de presa, anulando su función previa como desagüe de fondo.

Durante años se estuvo reflexionando sobre la necesidad o no de que la presa contara con un aliviadero. Siempre se había llegado a la conclusión de que no era necesario porque, al ser un embalse en derivación, las avenidas provenientes de su propia cuenca no agotarían el resguardo. Sin embargo, al ser una presa de materiales sueltos, clasificada en categoría A, el debate era recurrente.

Finalmente se decidió subsanar ambos problemas mediante la redacción y ejecución del proyecto de Gran Reparación de la presa de La Sotonera.

Este proyecto actúa en tres puntos principales, y varios accesorios, que se comentan a continuación.

4.5.1 *Desagüe de fondo*

La toma inferior, que antiguamente discurría en túnel a través de la presa como las demás, fue remodelada durante la construcción de la central hidroeléctrica de manera que ahora el agua circula en una tubería forzada de Ø3500 mm.

El nuevo desagüe de fondo consta de una tubería de Ø2500 mm que parte de un pantalón ejecutado sobre la tubería forzada, unos metros antes de la válvula de mariposa de entrada a la central hidroeléctrica. Esta tubería discurre enterrada durante 111,03 m hasta que llega a la cámara de compuertas donde hay dos válvulas compuerta tipo Bureau de asiento plano.

El sistema de aireación está formado por una tubería de aireación, una válvula compuerta y una ventosa de Ø400 mm para la compuerta de seguridad, y un sistema de aireación formado por tres tuberías y un deflector tipo tubo Peñarroya a la salida para la compuerta de regulación. El sistema se completa con un cuenco amortiguador que restituye el agua al cauce del río Sotón.

4.5.2 *Aliviadero*

La antigua toma superior de la presa de La Sotonera atravesaba la presa en túnel, y posteriormente contaba con un canal que transportaba el agua hasta su unión con la conducción proveniente de la toma intermedia, que acaba en el canal de Monegros. La toma superior ha sido eliminada en su totalidad, de manera que la obra de toma ha sido sustituida por un vertedero en labio fijo tipo Creager, en "pico de pato".

El aliviadero aprovecha los túneles de la antigua toma superior, pero el nuevo canal de descarga pasa en acueducto por encima de la conducción de la toma intermedia, hasta llegar a un cuenco de rotura que restituye el agua al cauce del río Sotón.

4.5.3 *Remodelación de la toma intermedia*

La anulación de las aportaciones al canal de Monegros de la toma superior, y la regulación por parte de la central hidroeléctrica de la toma inferior obligan a remodelar la toma intermedia para que sea

Figura 12. Vista de la salida del desagüe de fondo.

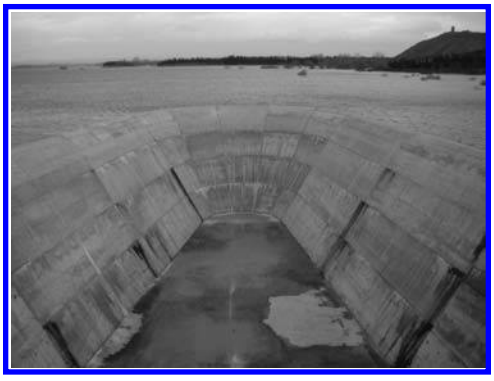

Figura 13. Vista del aliviadero.

Figura 14. Vista del edificio de toma de la Toma Inferior.

capaz de asegurar con total garantía los caudales que se deben aportar al canal de Monegros.

La toma intermedia constaba de seis compuertas deslizantes de regulación de 1,20 m de anchura por 2,80 m de altura, sin compuertas de seguridad, y que desembocaban en tres túneles que atravesaban el cuerpo de presa para suministrar caudales al canal de Monegros.

La remodelación de esta toma consistió en la instalación de compuertas tipo Taintor retráctiles en el interior de cada uno de los tres túneles, de manera que estas actúan como compuertas de regulación mientras que las antiguas compuertas deslizantes lo hacen como compuertas de seguridad. De esta manera se cuenta con doble cierre para garantizar la seguridad del suministro, así como de una mejor regulación.

4.5.4 *Otras actuaciones*

Junto con las actuaciones comentadas, también se ejecutaron otras de menor entidad, como la ejecución de viviendas, demolición de edificios antiguos o iluminación de puntos determinados de la coronación de la presa.

También se llevaron a cabo actuaciones complementarias de menor entidad, como asfaltado de caminos, protección de márgenes del río Sotón o limpieza del cauce.

4.6 *Proyecto de adecuación de la presa de La Sotonera al RTSPE*

Está previsto realizar una serie de actuaciones en la presa de La Sotonera para mejorar el funcionamiento y la seguridad de la presa. El proyecto cuenta con los apartados que se comentan a continuación.

4.6.1 *Remodelación de la toma inferior*

La toma inferior se utiliza como toma de la central hidroeléctrica, así como desagüe de fondo. Consta de un edificio de toma situado 45,30 m del eje de la presa, dentro del embalse.

Bajo este edificio existen 4 compuertas Stoney que prácticamente no se maniobran debido a problemas surgidos por la falta de mantenimiento, imposible de ejecutar ya que no puede realizarse el mantenimiento del eje del vástago de accionamiento ni se tiene acceso al emplazamiento de las compuertas.

En el proyecto se propone reparar o sustituir estas compuertas de manera que queden plenamente operativas. También se propone sustituir el accionamiento, actualmente eléctrico, por otro oleohidráulico.

4.6.2 *Remodelación de la toma intermedia*

La toma intermedia tiene un problema similar al de la toma inferior. Actualmente está regulada por las compuertas Taintor que se colocaron en la obra de Gran Reparación.

Consta, como la toma inferior, de un edificio de maniobras situado dentro del embalse. Bajo este edificio se encuentran las compuertas de seguridad, que son seis compuertas deslizantes. Estas compuertas no se encuentran plenamente operativas, por lo que se propone una sustitución de sus accionamientos eléctricos por otros oleohidráulicos para mejorar su funcionamiento.

4.6.3 *Adecuación de los emisarios*

La presa consta con un número de emisarios en el paramento aguas abajo para conducir el agua que se filtre por su espaldón, de manera que se puedan tener controladas las posibles fugas en la presa. Estos emisarios se encuentran en un estado de conservación no satisfactorio, observándose derrumbes en los cajeros, abundante vegetación y ausencia de camino de servicio para su mantenimiento.

Se propone su limpieza, reconstrucción en su caso y la construcción de un camino de mantenimiento y observación.

4.6.4 *Adecuación del paramento aguas arriba*
El talud está protegido del oleaje mediante un recubrimiento de losas de 0,50 m de espesor, de arenisca u hormigón según las zonas que desciende desde la coronación (cota 419,60) hasta la cota 412,00 m. Esta protección va apoyada sobre una capa de grava gruesa y está rematada por un pretil bateaguas.

Debido a la acción del oleaje se observa falta de alineación en zonas localizadas de losas de 0,50 m de espesor en el talud comprendido entre las cotas 419,60 y 414,09 m.

Se propone un saneo de estas zonas, mediante la extracción y acopio de las losas de hormigón, relleno y compactación con grava y posterior colocación de las losas.

4.6.5 *Adecuación del paramento aguas abajo*
El talud aguas abajo de la presa está configurado por 3 secciones y compuesto en su totalidad por gravas compactadas. El estado de estos taludes es satisfactorio y la resistencia de los taludes a los agentes atmosféricos está confiada con éxito a la vegetación. No obstante se observa la implantación de especies arbóreas (pinos) que ocasionan por la acción erosiva de las raíces una disgregación de las gravas con el riesgo correspondiente de que un vendaval pueda desarraigarlos y de esta forma originar desperfectos en el talud.

La solución consiste en el arranque, destoconado y posterior relleno del hueco ocupado por las raíces con grava compactada. La actuación se realizará en todo el arbolado situado en los taludes y en una franja longitudinal de 10 m de anchura a lo largo del pie de la presa de aguas abajo.

Paralelamente, se realizará el desbroce limpieza y reperfilado de todas las superficies de las bermas del paramento aguas abajo, compactación superficial de las gravas, extensión de un geotextil anticontaminante y que evite la aparición de vegetación y extensión de una capa de 25 cm de zahorra compactada sobre la que se pueda circular para facilitar las operaciones de mantenimiento.

4.6.6 *Adecuación del camino de coronación*
El camino de coronación de 4,30 m de anchura está asfaltado y tiene una acera de 0,70 m de anchura junto al pretil y un arcén con bordillo de 0,90 m de anchura. El pretil tiene 0,80 m de altura y 0,40 m de anchura y cada 25 m dispone de un refuerzo de sección cuadrada de 1,25 m de altura y 0,60 m de espesor.

En la zona pavimentada se observan blandones localizados debidos al asentamiento del núcleo central de arcilla. El pretil de coronación tiene una altura, 0,80 m, que resulta pequeña por motivos de seguridad. Además el camino cuenta con una instalación de alumbrado insuficiente.

Se propone la reparación de blandones, que comprende la excavación hasta 1,50 m de profundidad mínima en un perímetro que comprende la superficie total del blandón más 1 m de holgura en todo su perímetro. Compactación de fondo de excavación, relleno compactado de arcilla en tongadas de 0,20 m de espesor hasta el 98% PN, construcción de base granular de zahorra compactada de 30 cm de espesor hasta el 98% del P.M. y solera de hormigón superficial de 25 cm de espesor con mallazo # 6/20.

Una vez realizada la reparación de blandones se propone el refuerzo del pavimento existente con la extensión de una capa de rodadura MBC-D 12 de 5 cm de espesor mínimo y pendiente transversal del 2% en dirección al pretil existente, además del drenaje del camino de coronación, que se realiza mediante tuberías PVC Ø110 mm a través del pretil hacia el talud aguas arriba disponiéndose 1 unidad cada 25 m de longitud.

El aumento de altura del pretil de coronación se resuelve con barandilla tubular de acero inoxidable situada encima del pretil anclada mediante placas a este y a las pilonas situadas cada 25 m.

Además, se prevé la instalación de alumbrado del camino de coronación mediante luminarias sobre columnas de 4 m de altura situadas en el arcén del camino cada 25 m, con sus correspondientes cuadros eléctricos.

5 CONCLUSIONES

El río Gállego es aprovechado principalmente como suministro de agua para riegos y abastecimientos. Un segundo uso es el hidroeléctrico, con numerosos aprovechamientos de este tipo. Se considera un tercer uso lúdico, muy importante en este río debido a la belleza de los parajes por los que discurre y a las características de ciertos tramos del río.

Las obras comentadas han sido necesarias o convenientes, y de su estudio se deduce que el aprovechamiento del río Gállego para su principal uso de riegos y abastecimiento es mejor, ya que se mejora la garantía de suministro y la calidad de la regulación, sin afectar a los demás usos del río.

Además, al mejorarse su regulación, se gana seguridad frente al comportamiento del río en caso de avenidas. También se consigue mayor seguridad en las tareas de explotación de las presas comentadas.

Por todo lo cual, se puede concluir que en la actualidad tenemos un río Gállego mejor regulado, más aprovechable y más seguro.

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Desarrollo del plan de puesta en carga de la presa de La Colada

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RESUMEN: La presa de La Colada está en España, en el río Guadamatilla, al norte de la provincia de Córdoba. El embalse con una superficie de 530 hectáreas a nivel máximo normal, ocupa terrenos en los municipios de Belalcázar, El Viso e Hinojosa del Duque. En el año 2006 se finalizó la fase de construcción de la presa mediante la inyección de las juntas. En mayo de 2007 se procedió al cierre de las compuertas del desagüe de fondo, comenzando la puesta en carga del embalse. El comienzo de la explotación se prevé en un futuro inmediato, en cuanto estén terminadas las obras de conexión a los abastecimientos de la Zona Norte de Córdoba y a Almadén y su Comarca. Durante el llenado, el comportamiento de la presa en los distintos escalones de carga se ha analizado haciendo uso de la información facilitada por los elementos de auscultación. Se han observado los péndulos directos e inversos, los caudales de filtración, las temperaturas del aire máxima y mínima, la precipitación total diaria y el nivel de embalse.

ABSTRACT: La Colada dam is situated in Spain, on Guadamatilla River, at the north of Cordoba. The reservoir with an area of 530 hectare for the highest normal level, hold the lands of Belalcazar, El Viso and Hinojosa del Duque. On 2006 finished the construction of the dam by the joint inyection. On may 2007 we proceed to close the bottom gates, beginning at this time the origin of beginning the putting in load of reservoir. The beginning of the exploitation is foreseen in an immediate future, when the works of connection are finished to the supplies of Zona Norte de Córdoba and Almadén y Comarca. During the filling, the behavior of the dam in the different steps of load it has been analyzed using the information facilitated by the monitoring elements. It have been observed the direct and inverse pendulums, the flows of filtration, the temperatures of the air maximum and minimal, the total daily rainfall and the level of reservoir.

1 INTRODUCCIÓN

La presa que origina el embalse de La Colada se ubica en el río Guadamatilla a unos 3 km aguas abajo de la confluencia de los ríos Guadamatilla y Guadarramilla. Está formado por un brazo principal sobre el río Guadamatilla de longitud 8.990 m en dirección sur-norte. El río Guadarramilla genera un segundo brazo en la margen derecha del cauce, de dirección aproximada este-oeste, de longitud 2.530 m. El embalse se sitúa al norte de la provincia de Córdoba, en la Comarca de Los Pedroches. La puesta en carga comenzó en mayo de 2007. Se programaron cinco escalones de carga para la fase de llenado para distintos niveles de embalse, así como cuatro desembalses parciales, distribuyendo la carga sobre la estructura y controlando en todo momento de una forma intensiva los equipos de auscultación de la presa.

El proceso de puesta en carga estaba previsto inicialmente con una duración de dos años, sin embargo, debido a la escasez de aportaciones en estos últimos años, el proceso de puesta en carga se ha tenido que prolongar casi un año más. Con las fuertes precipitaciones registradas desde mediados

Figura 1. Embalse en el tercer escalón de llenado.

Figura 2. Cierre del desvío del río.

Figura 3. Embalse lleno.

de diciembre de 2009 y durante enero y febrero de 2010 se han podido llevar a cabo las distintas fases del llenado y posteriores desembalses parciales.

El comportamiento de la presa en los distintos escalones de carga se ha analizado haciendo uso de la información facilitada por los elementos de auscultación. Tras alcanzar el escalón correspondiente se ha parado los tiempos estipulados para realizar un detenido examen de los resultados de las distintas lecturas de los elementos de auscultación. Se ha continuado con el siguiente escalón siempre con los resultados positivos. Se han tomado lecturas a lo largo de la puesta en carga, con frecuencia diaria, con una inspección detenida de las obras observando coronación de la presa, paramentos, desagües de fondo, estado de fábricas, laderas de la cerrada, etc. También se han observado los péndulos directos e inversos, los caudales de filtración, las temperaturas del aire máxima y mínima, la precipitación total diaria y el nivel de embalse. Con carácter semanal se han medido los caudales de filtraciones, las subpresiones, los movimientos en fallas (extensómetros en estribo izquierdo), los medidores internos electromagnéticos de apertura de juntas, las bases tridimensionales de control de desplazamientos entre bloques, los extensómetros de cuerda vibrante y rosetas extensométricas y la temperatura de agua en embalse. Además de forma mensual se ha realizado una nivelación y colimación en coronación, en todos los bloques.

2 CARACTERÍSTICAS DEL EMBALSE

La presa es del tipo arco-gravedad, con una longitud de 274 m en planta y una altura máxima sobre cimientos de 48,50 m. La directriz curva tiene un radio de 149 m en la sección correspondiente al eje de la presa. En el cuerpo de presa se distinguen dos secciones tipo diferentes, las correspondientes a los estribos y al aliviadero. La sección tipo de los estribos, coronada a la cota 478, presenta el paramento de aguas arriba vertical. El de aguas abajo se hace vertical por encima de la cota 474,50 y, entre las cotas 466,30 y 474,50, se dispone con talud 0,1/1. Desde esta última cota hasta cimentación presenta un talud 0,5/1. La sección tipo del aliviadero presenta, paramento de aguas arriba vertical y talud de 0,5/1 en el paramento de aguas abajo, a partir de su tangencia con el perfil Creager de vertido. La coronación de la presa tiene una anchura uniforme de 5 m, con dos aceras de 1,50 m de ancho.

El volumen de embalse a cota 473,50 correspondiente a N.M.N. es de 57,69 Hm^3. La cuenca de aportación tiene 395,15 Km^2 de superficie, con una aportación anual media de 32 Hm^3. La presa se ha clasificado en función del riesgo potencial como B.

3 GEOLOGÍA

El emplazamiento del embalse se encuentra a unos 6 km al norte del pueblo de El Viso (Córdoba) en el río Guadamatilla, en el llamado "Cerro de La Colada", en Los Pedroches.

La zona de la cerrada donde se ubica la presa, (en ella estuvo el Molino de la Colada), corresponde a rocas del Paleozoico, grauvacas. Estas rocas sufrieron las acciones correspondientes al plegamiento Hercínico, fracturas y metamorfización.

Como sucede en la zona circundante, en la cerrada se observan tres familias de fracturas. Una primera y más antigua tiene dirección N-150º-E, que es la dirección de los principales pliegues de la zona. A ella queda asociada la intrusión ígnea del Batolito de los Pedroches pues, a favor de estas fracturas, se produjo la intrusión de diques de pórfidos graníticos y grano dioríticos.

La constitución geológica del vaso de embalse corresponde en su mayor parte a estas granodioritas. Solo la parte del vaso cercana a la presa es de rocas Paleozoicas (grauvacas, etc.) que han experimentado cierto metamorfismo.

La mayoría de las rocas que cubrirán el embalse ha sufrido las acciones del Plegamiento Hercínico, y como consecuencia de ellas se produjeron varias familias de litoclasas. Estas quedan más abiertas, como consecuencia de la decompresión, en las capas próximas a la superficie y por ellas ha pasado agua generando un proceso químico de meteorización.

En cambio, a partir de cierta profundidad, las masas rocosas mantienen su impermeabilidad original.

El collado situado en la margen izquierda, fue impermeabilizado durante la ejecución de las obras, por lo que, aparte de este, no hay ningún lugar, cercanía de collado etc, donde fracturas algo abiertas puedan poner el agua del embalse en comunicación con zonas externas. En consecuencia, puede afirmarse que, a efectos prácticos, la totalidad de embalse es impermeable.

Es de resaltar que, las fallas originadas por movimientos tectónicos en esta zona, fueron rellenadas por intrusiones filonianas, que restituyeron al macizo rocoso su impermeabilidad original.

En ninguna parte del vaso se ha visto que haya posibilidad de deslizamiento de laderas con magnitud que entrañe peligro. La razón de ello es que, dejando aparte los cauces de los ríos Guadamatilla y Guadarramilla, en los sitios donde se ha concentrado material con cohesión nula o muy reducida, los espesores no sobrepasan los 2 m (comprobadas con calicatas) y además las pendientes son suaves.

Para la supervisón del estado de la cimentación de la presa se han realizado visitas periódicas a las obras cuando la excavación de cada uno de los bloques había llegado a la cota prevista y se había realizado la limpieza del escombro procedente de la excavación.

Estas visitas se realizaron entre los meses de octubre de 2003 y marzo de 2004. Este periodo fue el necesario para que se alcanzaran sucesivamente los fondos de excavación de cada uno de los bloques y para que cada uno de ellos estuviera limpio y en unas buenas condiciones de observación.

El macizo rocoso sobre el que se apoya la presa está dividido en 5 sectores o zonas que corresponden con el estribo derecho, el estribo izquierdo, la parte derecha del fondo del valle, la parte izquierda del fondo del valle y la zona central del valle. Esta división se ha realizado intentando homogeneizar las características observadas y las condiciones de cimentación.

El macizo rocoso esta formado por una roca de metamorfismo térmico generado al favor de las intrusiones graníticas próximas. La roca sobre la que se apoyará la presa es una metagrauvaca de color gris azulado y ocasionalmente marrón grisáceo. Acompañando a esta litología que permanece homogénea en prácticamente toda la cimentación aparece en el centro del valle un dique de grandes dimensiones de granito leucocrático con grandes fenocristales rectangulares. El dique de granito tiene un espesor real de al menos unos 15 m en la zona de apoyo de la presa. De manera ocasional y puntualmente aparecen unos diques de composición cuarcítica de color blanco y que en función de su proximidad al dique granítico central parece acercarse a una composición aplítica.

El macizo rocoso se encuentra intensamente fracturado por varias familias de diaclasas que en zonas pueden llegar a ser hasta 6. Este gran número de juntas es relativamente poco habitual. En general se puede decir que se trata de unas juntas o diaclasas muy continúas o persistentes con un espaciado entre 0,60 y 2,0 m y con una rugosidad muy baja.

Del análisis de las diaclasas lo primero que llama la atención es la existencia de dos grupos de familias, unas subverticales con altos buzamientos y otras con buzamientos menores de 50º e incluso subhorizontales. Ambos grupos se analizaron en conjunto y no se han separado para su caracterización.

De manera resumida se puede interpretar que han existido al menos dos fases de deformación geológica en el entorno del emplazamiento de la presa. La primera genera un sistema de diaclasas que posteriormente es girado por una segunda fase de deformación horizontalizando las primeras diaclasas. La segunda fase de deformación produce la intrusión del granito y genera el sistema de diaclasado más vertical.

Analizando los datos de orientación y comparando los pertenecientes a cada uno de los estribos se observa que unas familias mantienen su orientación constante mientras que otras sufren una variación del orden de los 30º. Este cambio de orientación parece estar asociado a la intrusión del dique de granito central.

Además de la existencia de un diaclasado intenso se ha detectado la presencia de varias fallas

Figura 4. Embalse durante el cuarto escalón de llenado.

durante la realización de la cartografía geológica de los planos de cimentación de los bloques. Estas fallas se han numerado de la 1 a la 16 y están correctamente posicionadas por datos topográficos. La posición a lo largo de los planos de cimentación de la presa se puede observar en la planta geológica y en el perfil geológico que se presenta en los planos nº 3 y 4.

A modo de resumen y de forma esquemática se trata de un macizo rocoso de calidad R.M.R. muy mala, intensamente diaclasado en el que se identifican hasta 6 familias de diaclasas de baja rugosidad bajo espaciado y elevada continuidad o persistencia. Alguna de estas familias tiene una orientación muy desfavorable respecto a la dirección de la excavación y de las laderas, especialmente en el estribo izquierdo.

La meteorización general de la roca es de grado II y la resistencia media es de grado 4.

En el estribo izquierdo entre los bloques 9 y 11 aparece una zona con grado de meteorización III y resistencia 1. Esta zona se prolonga bajo la cimentación de los bloques 13, 15 y 17. Las características geotécnicas de esta zona no resultan concordantes con el resto del material.

En el eje de la presa existe un dique de intrusión granítica de 15 m de potencia. A lo largo de toda la cerrada se han identificado un total de 5 diques de cuarzo.

Se han definido un total de 16 fallas situadas topográficamente a lo largo de la cerrada. Estas fallas presentan un relleno arcilloso con fracción brechoide de espesor variable que oscila entre 2 y 5 cm. Las fallas definidas son las observadas en la excavación, pero viendo el estilo de fracturación del macizo rocoso mediante los perfiles geológicos realizados es fácil suponer una repetición sistemática en el interior del macizo.

Especialmente en el estribo izquierdo existen una serie de fallas subhorizontales que generan planos de debilidad del macizo rocoso con posibilidad de movimiento hacia el eje del valle. Estos planos de debilidad se sitúan bajo la cimentación de los bloques de la presa en este estribo.

Las fallas a su vez pueden ser direcciones preferentes en la circulación de agua.

Se ha realizado una campaña de inyecciones de consolidación del terreno de apoyo de la presa. Son cinco alineaciones de taladros paralelas al eje de la presa en el entorno del cauce, reduciéndose a dos alineaciones en estribos. La distancia entre cada dos alineaciones es de 5 m, siendo ésta también la distancia entre dos taladros contiguos, los cuales quedan dispuestos al tresbolillo.

A lo largo del contacto de la presa con el terreno de apoyo se ha previsto la realización de una pantalla de impermeabilización, para impedir eventuales filtraciones y disponer de un medio

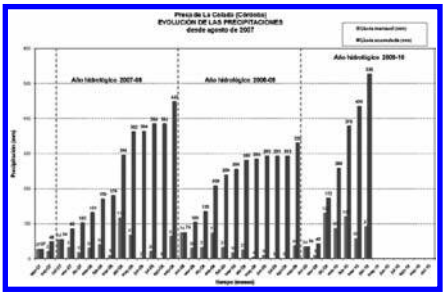

Figura 5. Evolución de las precipitaciones.

para controlarlas. Los taladros se perforan cada 5 m aproximadamente.

Entre cada dos de éstos, se sitúa un nuevo taladro que, por tanto, quedará a 2,50 m de los taladros contiguos. Estos nuevos taladros se introducen, como los anteriores, una profundidad mínima de 20 m en roca.

Agua abajo, a una distancia de 1,5 m de esta pantalla de impermeabilización, se sitúa una pantalla de drenaje para el control de las filtraciones y de las subpresiones de la presa. La pantalla de drenaje está constituida por taladros verticales alineados, con 5 m de separación entre sí y que penetran, al menos, 20 metros en la roca de cimentación.

Por otro lado se ha llevado a cabo un tratamiento de consolidación del estribo izquierdo. Este tratamiento se ha realizado con objeto de controlar y reducir las inestabilidades que se deben a la existencia de la serie de fallas subhorizontales que generan planos de debilidad del macizo rocoso con posibilidad de movimiento hacia el eje del valle. Es la presencia de estas fallas, que a su vez pueden ser direcciones preferentes en la circulación de agua, las que aconsejaron realizar un tratamiento complementario al inicial.

La consolidación se llevó a cabo mediante la perforación (y posterior inyección) de una serie de taladros con las mismas condiciones generales que el procedimiento del tratamiento ejecutado durante las inyecciones de consolidación ya existentes.

La consolidación del estribo consistió en 317 taladros de 5 m de profundidad en roca y un diámetro mínimo de dos pulgadas, resultando una longitud total de unos 3.409,4 metros de perforación (unos 1.342,4 metros de perforación en hormigón y 2.067 m en roca).

Los taladros se ejecutaron a rotopercusión, con martillo en cabeza o de fondo.

4 HIDROLOGÍA

La presa de La Colada, de la que se incluyen a continuación las características físicas de su

cuenca de aportación, está situada en la cabecera del río Guadamatilla afluente del Zújar por su margen derecha:

– Superficie de la cuenca: 395 km^2
– Longitud del cauce principal: 37,3 km
– Pendiente media del río: 0,68%
– Tiempo de concentración: 7,30 horas

Se realizó una recopilación de los datos de las estaciones meteorológicas emplazadas en el interior de la cuenca del Guadamatilla o en sus proximidades y se seleccionaron aquellas cuyos datos tiene influencia en la misma, mediante el procedimiento de los polígonos de Thiessen.

Para tener una serie homogénea de datos en el período de tiempo que fue estudiado en Proyecto (1.951–1.993), se rellenaron y completaron las series con los datos obtenidos por la ecuación de regresión con la estación 4–273 por ser la más representativa de toda la cuenca por ocupar mayor superficie en los polígonos de Thiessen y tener las series de datos más completas.

Aplicando el método de los polígonos de Thiessen para determinar la influencia de cada una de estas estaciones en la cuenca, se obtuvo que la pluviometría media anual en la cuenca de La Colada es:

Tabla 1. Aportaciones al embalse de La Colada.

Estación	Superf (km^2)	Factor de peso	Aportación (mm)	
			Estación	Cuenca
4–267	7,85	0,020	459,24	9,12
4–271	74,75	0,189	564,10	106,71
4–272	26,65	0,067	448,36	30,24
4–273	76,40	0,193	501,27	96,92
4–274	47,75	0,121	475,07	57,41
4–275	36,80	0,093	494,52	46,05
4–276	24,00	0,061	492,69	29,92
4–277	28,00	0,071	507,40	35,95
4–278	49,85	0,126	519,93	65,59
4–288	5,40	0,014	477,75	6,53
4–291	1,77	0,045	363,60	16,29
Total	395,15	1,000		500,74

El río Guadamatilla no tiene estación de aforos propia, por lo que la evaluación de su aportación media anual se realizo en Proyecto utilizando los datos de cuencas afines que poseen estaciones propias de aforos y los datos pluviométricos medidos en estaciones meteorológicas. Para obtener la aportación media a partir de datos foronómicos se establece que esta es directamente proporcional a la superficie de la cuenca. Aplicando proporcionalidad entre las cuencas del Zújar y del Guadamatilla, tendríamos una aportación anual media de la Presa de La Colada de 32 hm^3.

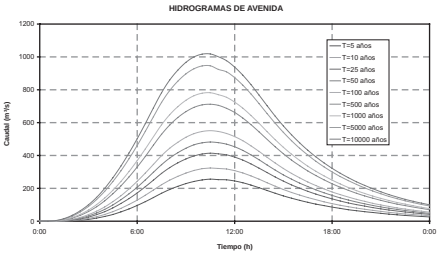

Figura 6. Hidrogramas de avenida.

Tabla 2. Hidrogramas de avenida del embalse de La Colada.

Periodo de retorno (años)	Precipitación (mm)	Caudal Punta (m^3/s)	Aportación (hm^3)
5	54,03	256	9,99
10	62,56	324	12,72
25	73,35	414	16,32
50	81,36	482	19,07
100	89,30	551	21,85
500	107,65	712	28,43
1.000	115,55	782	31,31
5.000	133,86	946	38,08
10.000	141,75	1018	41,02

Los caudales de avenida para distintos periodos de retorno son los indicados en la siguiente tabla.

4.1 *Aliviadero*

El aliviadero se sitúa en la zona central de la presa, abarcando los bloques 0, 1 y 2 completos y los bloques 3 y 4 parcialmente.

El vertedero consta de cinco vanos de labio fijo, de 10,75 m de longitud de vertido cada uno, con su umbral a la cota 473,50. El perfil de vertido es del tipo Creager, dimensionado para una altura de lámina de 2,50 m.

Las pilas se disponen con una anchura de 1,00 m agua arriba, disminuyendo radialmente ésta hacia el centro de replanteo de la presa. Los tajamares se diseñan redondeados para mejorar su hidrodinámica.

Los cajeros del aliviadero convergen radialmente hacia el centro de replanteo de la presa, reduciendo su separación desde los 58,00 m en la sección de vertedero hasta 48,52 m en la sección de entronque con el cuenco amortiguador.

La restitución al cauce se realiza mediante un cuenco amortiguador de 25,00 m de longitud, cuya anchura disminuye radialmente, desde los 48,52 m en su entronque con el aliviadero, hasta los 40,82 m en su sección final.

Los muros cajeros del cuenco son de hormigón armado y se coronan a la cota 445,0 que, tanto en la margen derecha como en la izquierda, se construyen en el pie de agua abajo de la presa.

La solera del cuenco amortiguador se diseña a la cota 435,50, con una elevación de 3,5 m de altura a su salida, coronada por dientes de 2,5 m separados entre sí 2,5 m. En la zona del cauce contigua se coloca una protección de escollera. En el canal de descarga del aliviadero, a la cota 447,176, se construyen unos deflectores a la salida de los desagües de fondo contra vertidos del aliviadero.

El paso de coronación de la presa sobre el aliviadero se resuelve mediante la construcción de un puente de cinco vanos apoyado sobre las pilas del mismo. La estructura la constituye una losa de vigas pretensadas por cada vano, de 1,00 m de canto y 3,50 m de anchura máxima, sobre la que se extiende el tablero, de 8,00 m de anchura, de los cuales 5,00 m corresponden a calzada y, los tres restantes, a sendas aceras de 1,5 m de anchura.

4.2 *Desagües de fondo*

Los desagües de fondo de la presa se sitúan en el bloque central de la misma (bloque 0).

Están constituidos por dos conductos rectangulares de 0,80 × 1,00 m, de acero inoxidable, que se desarrollan paralelos con una distancia entre sus ejes de 3,00 m.

La embocadura de los desagües se resuelve mediante una estructura de hormigón armado que sustenta una reja desmontable, de acero inoxidable, que permite el acceso a dicha embocadura desde el interior del embalse.

El umbral de la sección de embocadura se sitúa a la cota 447,50 y el techo de la estructura, a la 448,85. El eje de los desagües se sitúa a la cota 442,50 m. El embalse muerto de 0,36 Hm^3.

Los desagües de fondo se cierran mediante dos compuertas rectangulares deslizantes cada uno, tipo Bureau, alojadas en una cámara de válvulas dispuesta en el interior del cuerpo de presa, con acceso desde ambas márgenes a través de la galería de visita en el interior de la presa.

Las compuertas disponen de dispositivo de aireación, mediante una conducción de 300 mm de diámetro que comunica con un colector común de 600 mm de diámetro que sale al exterior por uno de los cajeros del aliviadero y de los correspondientes by-pass para igualar presiones internas en apertura. Así mismo, disponen de fines de carrera e indicadores de posición sensorizados.

La restitución del agua al cauce se realiza mediante salida al cuenco amortiguador del aliviadero, anteriormente descrito.

La cámara de válvulas tiene unas dimensiones, en planta, de 5,00 × 7,00 m, con una altura máxima de 4,45 m. Su solera se dispone a la cota 444,50.

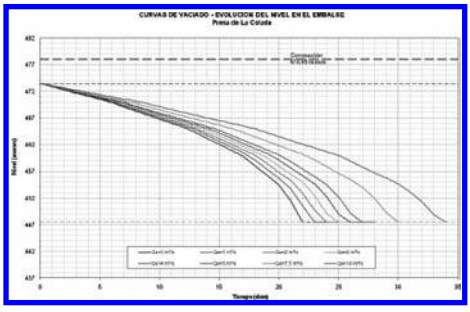

Figura 7. Curvas de vaciado del embalse.

Por otra parte, en la cámara se recogen las filtraciones procedentes de la galería de visita y se evacuan éstas al cuenco amortiguador, previo aforo de las mismas mediante aforadores tipo Thomson en los finales de las canaletas de desagüe dispuestas en la solera de la misma.

A partir de las leyes de descarga definidas para el desagüe de fondo y tomas, y de la curva característica del embalse, se han determinado los tiempos necesarios para los vaciados totales y parciales del embalse cuando los caudales entrantes varían de 0 a 10 m^3/s. Se ha representado gráficamente la relación entre el nivel de embalse y caudales entrantes, con estos tiempos de vaciado.

Estas curvas permiten que se pueda conocer para cada situación concreta de llenado la capacidad de control que se tiene sobre el nivel de embalse, ya que determina el tiempo que se necesitaría en caso de que, en un momento determinado, se precisase realizar un vaciado total o parcial del agua almacenada.

5 ESCALONES DE LLENADO

5.1 *Primer escalón de llenado*

El primer escalón de llenado se establece para una altura máxima del 28,77% de la máxima carga, es decir, 10,5 m (cota 447,50 msnm). Supone llenar el embalse hasta la cota de entrada en la torre de toma del desagüe de fondo, por lo que la velocidad de ascenso será en todo momento la definida por el propio río, ya que no es posible regularla con los órganos de desagüe de la presa.

Este escalón de carga supone someter a la presa a un empuje del 8,28% del empuje a NMN. El empuje ejercido sobre la presa es de 55,13 toneladas por metro lineal. Si bien la carga aplicada es muy reducida, a partir de este punto comienza a entrar en carga los bloques en los estribos. Debido al estado de los mismos, es conveniente reducir el ritmo de puesta en carga a partir de este punto. El volumen almacenado en este escalón es de 0,78 Hm^3.

Figura 8. Embalse tras las avenidas de febrero de 2010.

Una vez alcanzada la cota 447,50 msnm se ha mantenido durante un mes, regulando el nivel mediante descargas por los órganos de desagüe de fondo y toma del caudal ecológico cuando ha sido necesario. Durante todo el periodo se ha prestado especial atención a las posibles vías de filtración de agua que puedan aparecer, así como al nivel de eutrofización del embalse.

Al final de este periodo el Ingeniero encargado de la puesta en carga ha elaborado el correspondiente informe que recoge los datos de auscultación, análisis y posibles incidencias. Tras la redacción del informe se procedió al llenado del segundo escalón de carga.

5.2 *Segundo escalón de llenado*

El segundo escalón de llenado se establece para una altura del 50% de la máxima carga, es decir, 18,25 m (cota 455,25 msnm), limitando la velocidad ascensional a 0,5 m/día.

Lo indicado en el párrafo anterior debe entenderse como un valor indicativo medio, lo cual es indudablemente una recomendación orientativa. El Ingeniero Director de la Puesta en Carga, a la vista de cómo se fueron produciendo las entradas de agua al embalse, y la consiguiente subida de nivel, decidió la cuantía en que procede realizar vaciados parciales para acomodarse en lo posible a la velocidad de llenado antedicha.

Este escalón de carga supone someter a la presa a un empuje del 33,3% del empuje a NMN. El empuje ejercido sobre la presa es de 166,50 toneladas por metro lineal. Con esta carga es de esperar un ajuste tensional y de deformaciones además de comprobar la impermeabilidad tanto de la presa como de su cimentación y de la cerrada.

El volumen almacenado en este escalón es de 4,40 Hm^3, siendo el volumen total hasta este punto de 5,18 Hm^3.

Una vez alcanzada la cota 455,25 msnm se mantuvo durante un mínimo de 1 mes, regulando el nivel mediante descargas por los órganos de desagüe de fondo y toma del caudal ecológico si es necesario. Durante todo el periodo se prestó especial atención a las posibles vías de filtración de agua que puedan aparecer, así como al nivel de eutrofización del embalse, además de al movimiento en ambos estribos de la presa (bloques 6 y 7).

Tras comprobar que los niveles de eutrofización fueran aceptables y una vez alcanzada la cota 455,25 msnm sin realizar desembalses parciales, pasado el periodo establecido de mantenimiento de nivel se procedió bajar el nivel lentamente hasta alcanzar la cota 453,50.

Estaba previsto que, si en el transcurso de esta operación de desembalse se decidiese, por alguna razón justificada, bajar aun más el nivel y se llegara a vaciar totalmente el embalse hasta descubrir parcialmente las rejas de entrada, se debería ordenar que se proceda a la limpieza diaria y sistemática de éstas, para quitar las ramas y la maleza que queden atrapadas entre las pletinas, asegurándose de que, en caso de que sobrevenga una avenida, no quede la reja parcialmente taponada. Se recomendaba, en consecuencia, que esta operación de desembalse, si se opta por llevarla a cabo, se cometa durante el periodo de estiaje del río. En todo caso no fue necesario.

Una vez realizado el desembalse, en la cuantía establecida, se cerraron los desagües de fondo para recuperar el nivel hasta el máximo establecido (cota 455,25 msnm) comprobando que las medidas de auscultación se mantienen siempre dentro de la normalidad.

Al final de este periodo el Ingeniero encargado de la puesta en carga elaboró el correspondiente informe que recogió los datos de auscultación, análisis y posibles incidencias. Tras la redacción del informe se procedió al llenado del segundo escalón de carga.

5.3 *Tercer escalón de llenado*

El tercer escalón de llenado se establece para una altura de agua del 75% de la carga total, es decir, 27,375 m (cota 464,375 msnm), limitando la velocidad ascensional a 0,5 m/día, regulando la descarga por los órganos de desagüe de fondo y toma del caudal ecológico si es necesario.

Este escalón de carga supone someter a la presa a un empuje del 56,25% del empuje a NMN. El empuje ejercido sobre la presa es de 374,7 toneladas por metro lineal. Con esta carga es de esperar un ajuste tensional y de deformaciones, pero esta vez con movimientos ya detectables en la presa. El volumen almacenado en este escalón es de 16.12 Hm^3, siendo el volumen total hasta este punto de 21,3 Hm^3.

Una vez alcanzada la cota 464,38 msnm se debía mantener durante un mínimo de 1 mes, regulando el nivel mediante descargas por los órganos de desagüe de fondo y toma del caudal ecológico si fuera necesario. Ello no fue posible debido a las

Figura 9. Embalse en el cuarto escalón de llenado, antes del primer vertido por el aliviadero.

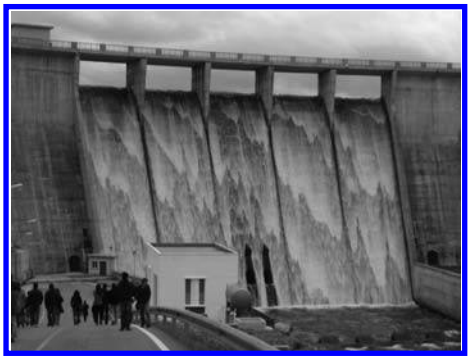

Figura 10. Vertido del aliviadero.

continuas lluvias registradas desde mediados de diciembre de 2009. En este escalón se debía controlar la evolución de los movimientos tanto de la presa y su cimiento como de ambos estribos. Para ello se realizaron lecturas de los extensómetros de varillas, péndulos, medidores internos electromagnético de apertura de juntas y bases tridimensionales de control de desplazamientos.

Por otro lado se comprobó la evolución, durante el escalón de carga, de las presiones intersticiales en el cimiento mediante la lectura de los piezómetros de cuerda vibrante.

Por último, se llevó un control de los caudales, en cantidad y evolución a lo largo del proceso, aforados por filtraciones en los aforadores Thomsom y en las posibles nuevas vías de agua detectadas.

En este caso se sobrepasó la cota 464,38 msnm sin realizar desembalses parciales por lo que se decidió proceder a la carga del siguiente escalón.

Sin realizar el desembalse correspondiente, se comprobó que las medidas de auscultación se mantienen siempre dentro de la normalidad.

5.4 *Cuarto escalón de llenado*

El cuarto escalón de llenado se establece para una altura de agua del 90% de la carga total, es decir, 32,85 m (cota 469,85 msnm), limitando la velocidad ascensional a 0,3 m/día.

Este escalón de carga supone someter a la presa a un empuje del 81% del empuje a NMN. El empuje ejercido sobre la presa es ahora de 539,56 toneladas por metro lineal.

El volumen almacenado en este escalón es de 18,6 Hm^3, siendo el volumen total hasta este punto de 39,91 Hm^3.

Una vez alcanzada la cota 469,85 msnm se mantuvo durante un mínimo de 1 mes, regulando el nivel mediante descargas por los órganos de desagüe de fondo y toma del caudal ecológico si es necesario. En este escalón se controló la evolución de los movimientos tanto de la presa y su cimiento como de ambos estribos. Para ello se realizaron lecturas de los extensómetros de varillas, péndulos, medidores internos electromagnético de apertura de juntas y bases tridimensionales de control de desplazamientos.

Por otro lado se comprobó la evolución, durante el escalón de carga, de las presiones intersticiales en el cimiento mediante la lectura de los piezómetros de cuerda vibrante.

Por último, se llevó un control de los caudales, en cantidad y evolución a lo largo del proceso, aforados por filtraciones en los aforadores Thomsom y en las posibles nuevas vías de agua detectadas. Cabe destacar que es a partir de este punto donde ya se han detectado vías de filtración en el bloque 8 (estribo derecho).

Los niveles de eutrofización han sido aceptables y se ha llegado a la cota 469,85 msnm sin realizar desembalses parciales, pasado el periodo establecido de mantenimiento de nivel se procedió bajar el nivel lentamente hasta alcanzar la cota 468,03 msnm.

Una vez realizado el desembalse, en la cuantía establecida, se cerraron los desagües de fondo para recuperar el nivel hasta el máximo establecido (469,85 msnm) comprobando que las medidas de auscultación se mantienen siempre dentro de la normalidad.

Al finalizar el ciclo de llenado se elaboró un nuevo informe suscrito por el Ingeniero encargado de la puesta en carga, antes de pasar al siguiente escalón de carga.

5.5 *Quinto escalón de llenado*

El último escalón de carga se establece para una altura de agua del 100% de la máxima, correspondiente a su máximo nivel normal (473,5 msnm, y 36,50 m de carga de agua). En este último escalón se limita la velocidad ascensional al 0,15 m/día.

Este escalón supone someter a la presa a un empuje del 100% del empuje y al 100% de las

Figura 11. Evolución del nivel de embalse 2009–2010.

tensiones máximas esperables en situación normal. Se controla la evolución de movimientos, prestando especial atención al comportamiento de ambos estribos y a los caudales de filtración y/o surgencias.

Se tiene especial cuidado en no sobreelevar el nivel de embalse por encima del nivel normal de explotación al maniobrar los órganos de desagüe.

En caso de que los niveles de eutrofización sean aceptables y se haya llegado a la cota 473,50 msnm sin realizar desembalses parciales, pasado el periodo establecido de mantenimiento de nivel se procederá bajar el nivel lentamente hasta alcanzar la cota 472,28 msnm.

Una vez realizado el desembalse, en la cuantía establecida, se cerrarán los desagües de fondo para recuperar el nivel hasta el máximo establecido (473,50 msnm) comprobando que las medidas de auscultación se mantienen siempre dentro de la normalidad.

Al finalizar el ciclo de llenado se elaborará un informe incluyendo todo el desarrollo de la puesta en carga, suscrito por el Ingeniero encargado.

5.6 *Previsiones de la puesta en carga*

La previsión de los tiempos teóricos necesarios para el llenado completo de la presa en los escalones establecidos en dos escenarios diferentes, con aportaciones medias del río y con las aportaciones medias del río en un año húmedo, entendiéndose por tal el año medio del 25% de los años de mayor aportación de la serie histórica.

Para el año medio, la puesta en carga se previó una duración de 3 años y 9 meses, mientras que si se presentan años húmedos, puede reducirse a 2 años y 2 meses.

5.7 *Cronología del llenado de embalse*

Las obras de la presa de La Colada concluyeron en el año 2005, aunque fue en 2006 cuando finalizó la construcción mediante la inyección de las juntas de la presa y el cierre del desvío del río. El 12 de abril de 2007 se procedió al cierre de los desagües de fondo mediante un acto oficial presidido por la Ministra de Medio Ambiente, con lo cual comenzó el llenado del embalse. La evolución del nivel de embalse y los eventos producidos durante todo el periodo de puesta en carga hasta la actualidad en que se alcanza la cota correspondiente al embalse lleno y último escalón del proceso de llenado corresponde a uno de los periodos más importantes en la vida útil de una obra de estas características.

El embalse se encontraba, en el momento del inicio de todo el proceso a la cota de umbral de entrada al desagüe de fondo, correspondiente al primer escalón del proceso de llenado. Este nivel inicial separa el embalse muerto, no utilizable, del embalse que posteriormente iba a ser utilizado para los múltiples fines que tiene la infraestructura.

El 14 de noviembre de 2008 se alcanzó el nivel correspondiente a este segundo escalón, y el 18 de diciembre de 2008, una vez verificado el correcto comportamiento de la presa, se procedió al desembalse parcial correspondiente, encontrándose el embalse al final de éste a la cota 457,25. El segundo escalón e inicio del tercero coincidió en el tiempo con la campaña de inyecciones realizada para el refuerzo de la impermeabilización y drenaje de los bloques centrales de la presa, que en ese momento eran los únicos que tenían su cimentación bajo el nivel de agua, dejando el resto para el momento en

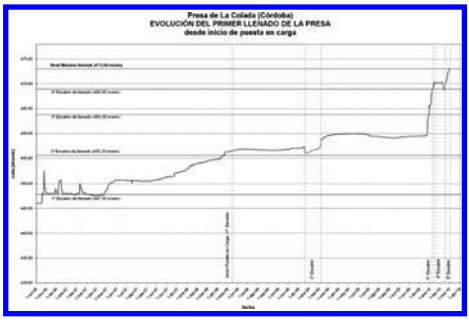

Figura 12. Evolución del primer llenado.

Figura 13. Evolución de los movimientos en los péndulos.

que tuvieran carga hidráulica y se pudiera evaluar la eficacia del tratamiento.

El cuarto escalón de llenado se alcanzó el pasado 30 de diciembre de 2009. Debido a las significativas aportaciones, se realizó una apertura de ambos desagües de fondo, durante 26 horas, cuando el embalse se encontraba a la cota 465,60. Debido a la fuerte sequía del año 2009, durante todo el mismo, no se superó la cota 460,00 hasta el reciente periodo de lluvias iniciado el 16 de diciembre, que produjo una subida del nivel de embalse de 10,84 m, con lo que se ha pasado directamente del 2º al 4º escalón, y de un volumen embalsado del 18,6% al 73,2%. El día 31 de diciembre una vez verificado el adecuado comportamiento de la presa y decidido que se continuase con el llenado para aprovechar las lluvias que se preveían en los siguientes días, se procedió al cierre de los mismos, posponiendo el escalón hasta que el nivel se estabilizase de forma natural al reducirse las aportaciones.

Desde el día 31 de diciembre, las lluvias continuaron, aunque de manera no muy intensa, si bien, la saturación existente de los suelos de la cuenca ha producido importantes aportaciones en los primeros días de enero que han supuesto que se alcance el cuarto escalón. El día 13 de enero de 2010, una vez terminados los periodos de lluvia más intensa, se alcanzó la cota 470,35, que sobrepasaba la correspondiente al cuarto escalón por lo que se procedió a abrir un único desagüe de fondo para mantener el nivel de embalse en la cota correspondiente al escalón durante un periodo de un mes, tal como marca el Plan de Puesta en Carga. Hasta el 9 de febrero se mantuvo la cota mediante reiteradas aperturas del desagüe de fondo o la toma ecológica, dado que en este periodo no han cesado las lluvias. El día 9 se inició el desembalse correspondiente al escalón, alcanzándose el 12 de febrero, la cota 468,70. A partir de este día, se inició el llenado correspondiente al quinto escalón.

El día 5 de marzo de 2010, debido a las continuas lluvias, que aunque no han sido muy intensas, han producido cuantiosas aportaciones debido a la saturación de la cuenca, se ha alcanzado la cota de aliviadero, correspondiente al Nivel Máximo Normal y 5º escalón de llenado. Por lo tanto actualmente el embalse ha alcanzado su nivel máximo, cota 473,50, que corresponde al quinto escalón de puesta en carga y el 100% de capacidad.

Figura 14. Vertido del aliviadero y desagüe de fondo.

Figura 15. Vertido del aliviadero.

Figura 16. Vista de la presa desde agua abajo.

6 CONCLUSIONES

El comportamiento de la estructura ha sido correcto durante todo el proceso de puesta en carga. Hay que reseñar la presencia de filtraciones en la junta 3–5 del estribo izquierdo, en la parte más alta de la presa, a nivel de la galería superior. Actualmente se está realizando una campaña de inyecciones en el cimiento y para la corrección de las filtraciones detectadas, así como la reperforación de drenes.

En la ladera izquierda, recorriendo todo el estribo de la presa, se perforaron inicialmente durante la ejecución de las obras, tres pozos de recogida de filtraciones, con una serie de perforaciones subhorizontales en abanico para interceptar posibles filtraciones en el pie de presa. Hasta el momento solo han detectado filtraciones de la propia ladera, sin que se hayan visto filtraciones que provengan de la presa.

Concerning reservoir capacity, water resources management and hydraulic performance

Outlet works

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Googong Dam spillway—remedial works to repair extensive rock erosion

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ABSTRACT: A series of relatively small floods caused extensive rock erosion, approximately 5,000 m³, in the unlined section of the spillway channel at Googong Dam. A range of protective remedial works were installed during the 1980s with varying success. The most recent phase of work commenced in 2006 with a review of the spillway's performance, assessment of future erosion potential and a comparison of remedial works options. The detailed design was developed for the preferred option, comprising the retro-fitting of a concrete-lined chute, the raising and extension of the spillway chute walls and strengthening of other existing components. Construction is currently underway by the Bulk Water Alliance and is due for completion in late 2010. This paper will present details of the history of this project including the initial assessment, review of rock erosion potential, options comparison and the detailed design. Key challenges overcome during the construction of these works will also be discussed.

1 INTRODUCTION

Googong Dam was completed in 1978 and forms one of the main water supply storage reservoirs for the Australian Capital Territory (ACT) and surrounding region. It is a 66 m high zoned earth and rockfill embankment on the Queanbeyan River and has been classified as an extreme consequence category dam in accordance with the guidelines in DSC (2002). The spillway through the right abutment consists of:

- a quarry forming the curved approach channel;
- an uncontrolled ogee crest with a crest length of 124 m;
- a 64 m long concrete-lined chute which converges to 64 m at the downstream end;
- an 80 m long unlined chute in rock with a width of approximately 64 m; and
- a stepped drop of approximately 24 m to the downstream quarry which forms the plunge pool.

Successive flood events in 1978 and through the 1980s resulted in extensive erosion in the unlined section of the spillway chute, including a large erosion hole, up to 19 m deep and 25 m wide, in the upper part of the unlined spillway chute and an exit channel downstream of the hole. The erosion hole and eroded exit channel are known as the "amphitheatre" and "canyon", respectively. Staged remedial works were undertaken in the 1980s to protect the eroded structure. The pre-construction condition of the spillway chute in 2008 is shown in Figure 1.

GHD Pty Ltd was engaged in 2006 to undertake a review of the historical performance and prediction of future performance of the structure. This engagement included review and assessment of previous work, the development and comparison of options and ultimately the detailed design of the preferred arrangement.

The construction of the remedial works is being undertaken as a construction only package by the Bulk Water Alliance (BWA) involving ACTEW Corporation and ActewAGL (the Owner Partners), GHD (the Design Partner) and Abigroup and John Holland (the Construction Partners). Construction of the remedial works is due for completion in late 2010. Other packages included

Figure 1. Aerial view of spillway from downstream in 2008.

Figure 2. Initial development of the "amphitheatre" after discharging 230 m³/s in September 1978.

in the Alliance are the design and construction of the Enlarged Cotter Dam (ECD), an inter-basin transfer pipeline from the Murrumbidgee River to Googong Dam (M2G) and other bulk water assets.

2 REVIEW OF ROCK EROSION AND COMPARISON OF REMEDIAL WORKS OPTIONS

2.1 *Assessment methodology*

The assessment of the performance of the spillway included the following stages:

- review of the historical performance including past flooding and remedial works;
- review of the geological setting;
- identification of possible failure mechanisms;
- assessment of erodibility of the unlined spillway section; and
- development and comparison of remedial works options to address identified failure mechanisms.

A concept design was developed for the preferred option based on the outcomes of this assessment process and the detailed design was produced as a second stage of the project.

2.2 *Historical performance*

Development of the erosion in the unlined portion of the spillway chute commenced during the first flood event in June 1978. The extent of the erosion was significantly increased during the September 1978 flood (230 m³/s with an Annual Exceedance Probability (AEP) of greater than 1:10). This is shown in Figure 2 which is a view looking from right to left across upper section of unlined spillway chute with the "amphitheatre" erosion clearly evident in the centre of the photo.

The 1984 flood event, discharging up to 265 m³/s, significantly widened and deepened the erosion hole. Remedial works including the construction of anchored concrete steps on the upstream face of the "amphitheatre" were constructed in 1986–7, but further significant erosion occurred in July 1988 during the flood of record (470 m³/s with an AEP of less than 1:20). This is shown in Figure 3, looking from left to right across the "amphitheatre".

As part of the spillway capacity augmentation works in 1989–91, which included post-tensioned anchoring of the spillway crest structure and a 4.5 m raising of the dam crest with a mechanically stabilised earth section, further protection works were constructed in the spillway, including:

- construction of concrete steps on the left and right side of the "amphitheatre";
- placement of a concrete slab in the base of the "amphitheatre";
- installation of two rows of passive anchors across the chute downstream of the end of the concrete-lined section as an erosion cut-off; and
- shotcreting, anchoring and placement of concrete protection in the unlined section of the spillway.

Shortly after completion of these remedial works in 1991, the spillway safely passed a peak discharge of 410 m³/s.

Since the mid-1990s, there have been no significant flood events, but ongoing deterioration by rock toppling downstream of the "amphitheatre" has occurred independent of spillway operation.

Figure 3. Steps constructed as part of 1986–87 remedial works and widened "amphitheatre" following July 1988 flood.

Figure 4. The pre-construction condition of the "amphitheatre".

The pre-construction condition of the "amphitheatre" area is shown in Figure 4. The volume of rock eroded to date is about 5,000 m³.

2.3 *Geological setting*

2.3.1 *General*

The spillway area can be divided into two lithological domains as shown in Figure 5 and is briefly summarised as follows:

- The upstream part of the spillway is underlain by felsic extrusive rocks referred to as dacite, which is highly jointed and fractured throughout, with several zones being identified as "sheared zones", and also "crushed seams" and "altered/weathered zones"; and
- The rock mass currently exposed in the downstream part of the unlined section of the spillway chute is predominantly a granitic rock which is more regularly jointed.

The strongly altered contact between the dacite and the granite generally coincides with the downstream end of the "amphitheatre" (as shown in Figure 3), and the eroded hole or "amphitheatre" is largely contained within the dacite. The "canyon" has formed downstream of the "amphitheatre" within the granite along a steeply dipping fracture zone on the left side of the spillway channel which strikes sub-parallel to the spillway axis. Similar erosion has commenced in a second fractured zone with a similar orientation on the right side of the spillway.

Figure 5. Simplified plan of site geology (Image source: Google Earth Pro, accessed 12 August 2008).

2.3.2 *Geological factors in dacite erosion*

The highly fractured/jointed and weathered state of the dacite rock mass indicates that this material would be subject to rapid erosion even under low to moderate flow energies. It was assessed that such flow energies would pressurise the very closely spaced fractures/joints/seams within this rock mass, leading to rock fragments being jacked out. The development of the "amphitheatre" scour hole is graphic evidence of the erodibility of this material.

It was identified that the presence of "sheared zones" and "crush seams" within this rock mass would lead to concentration of erosive processes under low flow conditions. The concrete/shotcrete protection, placed at various times and with varying quality control, could not be relied on to protect the underlying rock for all flow conditions.

The severe past erosion experienced within this rock material supported a conclusion that this material should not be exposed to even low levels of flow energy. Remediation strategies must prevent flow access to this material, and ensure the existing protection is capable of resisting all predicted flow conditions to protect the existing spillway lining and crest structure.

2.3.3 *Geological factors in granite erosion*

Although the granite rock mass was considered to be far less erodible than the upstream dacite rock mass, the existence of unfavourably oriented

persistent joints and fracture zones have led to marked erosion under moderate flow energies.

Back-cutting erosion has been experienced within the granite concentrated along the fracture zones that trend sub-parallel to the spillway axis (Figure 6). This erosion mechanism is most pronounced along the fracture zone located to the left side of the unlined chute (known as the "canyon"). It was considered that this erosion was accelerated by the "amphitheatre" focussing low to moderate flows to this side of the chute, accentuating back erosion along this fracture zone. Future erosion would undoubtedly concentrate in this area if the spillway was left in its present condition.

Back-cutting to a lesser extent has occurred along a parallel fracture zone located to the right side of the chute. It is possible that this area has been protected through the focussing of flow in the "amphitheatre" on the left side of the spillway. Previous geological mapping of this area prior to coverage by concrete indicates that this fracture zone becomes pro-gressively more developed upstream of the existing cut slope. It was anticipated that this fracture zone may also erode rapidly if exposed to flow energy.

The erosion of the "canyon" on the left of the chute has also removed lateral support to large blocks of rock separated by steeply dipping joints, striking sub-parallel to the spillway axis. The toppling failure of these blocks would likely be accelerated by flow erosion leading to progressive failure of the left hand side of the chute in this area.

The erosion mechanisms described above have been propagated by low level flows. The impact of higher level flows or even Probable Maximum Flood (PMF) flows would lead to very rapid erosion and removal of existing protection.

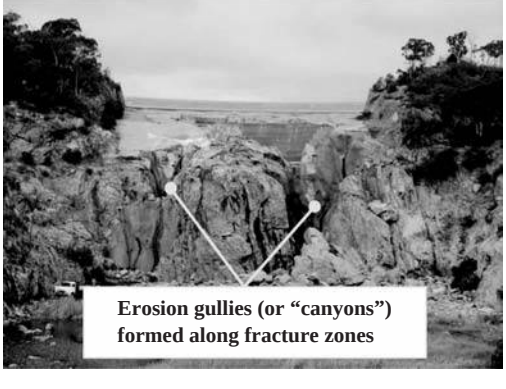

Figure 6. View from plunge pool looking upstream showing erosion concentrated in both right and left side fracture zones.

2.4 *Review of flood hydrology*

The PMF peak outflow estimate from the original design was 4,500 m³/s which was subsequently updated to 8,600 m³/s as part of the assessment of erosion and design of the 1989–91 remedial works (ACT Water, 1987). For the current engagement, the flood hydrology was reviewed, yielding a PMF peak outflow of 10,500 m³/s (SKM, 2007). This estimate was used in the assessment and development of options.

Based on this review, it was identified that the capacity of the spillway was insufficient to safely pass the PMF. The flood could be passed with minimal freeboard to the dam crest, but the walls in the existing lined chute would be overtopped by up to 4 m. Shotcrete protection had been placed against the rock above the existing walls during 1989–91 works and the adequacy of this was considered.

2.5 *Identification of high risk areas and possible failure mechanisms*

A site visit and workshop was held to identify high risk areas within the spillway area and possible failure mechanisms associated with these areas. Failure mechanisms identified included:

1. Erosion of the granite on the downstream left side of the "canyon" and in the "canyon" exposing the dacite to back-erosion.
2. Failure of the existing protection measures in the fractured granite zone on the downstream right side of the spillway allowing erosion of the granite and exposure of the dacite.
3. Failure of the concrete protection upstream of and surrounding the "amphitheatre" allowing erosion of the dacite.
4. Failure of the concrete protection in the "amphitheatre" exposing the dacite and sheared zone.
5. Failure of the existing lined section chute slabs;
6. Overtopping of the existing chute walls resulting in failure of the shotcrete protection and erosion of the underlying weathered rock.
7. Erosion of the right side quarry wall resulting in back erosion.
8. Failure of the facing panels of the mechanically stabilised earth section on the dam crest around the spillway crest from high velocity flow under PMF conditions.
9. Erosion of the unlined chute side walls resulting in back erosion.

2.6 *Erodibility assessment*

The resistance to erosion of the dacite was estimated using the method originally presented in Annandale (1995) and outlined in Wark et al (2004). The method involves determining the Erodibility Index (EI), a dimensionless parameter which is

based on the geological/geotechnical properties of the rock, and the power available in the stream flow for down-cutting and back-cutting. Rock properties relating to the strength and jointing (eg spacing, orientation, condition) are the main inputs in estimating EI. A threshold for the onset of erosion is defined in Annandale (1995) by an empirical relationship between EI and the stream power.

The stream power is related to the rate of energy dissipation which results from turbulence in the water flow. Turbulence at the boundary will produce pressure fluctuations which could initiate erosion by jacking, resulting in a unit of material being dislodged and displaced (Wark et al, 2004).

EI for the dacite was estimated from data included on logs for borehole within the spillway area. None of the boreholes for which information was available intersected the fractured zones. Therefore, the geological properties used in the estimation of EI for the material in the fractured zones were based on site observations.

From the geological properties and the derived EI, the threshold stream power (based on Annandale's empirical relationship) for the initiation of erosion in the dacite was estimated to generally be in the range of 200–500 kW/m^2. The threshold for the onset of the erosion in the sheared zones was estimated to be approximately 30 kW/m^2 or an order of magnitude lower than for the dacite.

Down-cutting and back-cutting were considered to be the critical hydraulic processes resulting in erosion. The power available to erode by down-cutting and back-cutting was calculated for flow into different depths of "holes". A "hole" may also result from flow over an undulation and impingement on the rock surface downstream of the undulation. The power available for down-cutting generally increases with depth of hole and for back-cutting generally decreases with depth.

The assessment indicated that flow over discontinuities and undulations with an asperity of about 0.5 m height would result in down-cutting in the dacite within the fractured zone. As the hole deepened in the sheared zones, it was found that the amount of energy being dissipated when the hole reached about 2 m to 3 m depth would be sufficient to initiate erosion of the intact dacite.

There was found to be sufficient energy available to back-cut at quite shallow depths in the intact dacite. The erosion mechanism was seen as one where down-cutting in the sheared zones combined with back-cutting and down-cutting in the intact dacite would rapidly widen any erosion hole which formed. A mechanism similar to this was considered to have caused the erosion observed in the "amphitheatre" and "canyon".

2.7 *Remedial works options development and comparison*

2.7.1 *Options from previous work*

The remedial works options presented in SMEC (2005) were used as a starting point for the development of further options. These options included:

- an "anchor & patch" option, similar to the approach adopted to date, involving application of shotcrete protection and installation of anchor bars on the basis that ongoing maintenance would be required following spillway operation;
- a "full fix" approach, including backfilling the "amphitheatre", retro-fitting a 70 m long reinforced concrete chute including a flip bucket; and
- an "intermediate" option, similar to the "full fix" except that the flip bucket was located approximately 35 m downstream of the end of the existing concrete-lined section.

The latter was proposed by SMEC as the preferred solution on the basis of the whole-of-life cost. It must be noted that any remedial works required in the existing lined section of the spillway were not included in the brief for the SMEC work.

2.7.2 *Options development and comparison*

The options development phase included the following tasks:

- The "anchor & patch" and "full fix" options proposed by SMEC were assessed in terms of the protection they provided against the identified failure mechanisms. Where it was considered an option may not fully protect against a possible failure mechanism, additional measures which may be required were identified. This process resulted in options termed as "enhanced".
- Additional remedial works were designed to concept level.
- Project cost estimates were developed.
- Relationships between spillway discharge and damage resulting from that event were developed.
- The Annual Risk Cost (ARC) for flood damage and the annual cost of Operation and Maintenance (O&M) of the structure were estimated.
- The total Net Present Cost (NPC) of the option was estimated as the sum of the project cost estimate and the NPC of the annual flood damage and O&M costs (using a service life of 150 years and assumed discount rates of 3% and 6%).
- The flood damage ARC was calculated by:
- Dividing the flood frequency curve into even increments (1 to n) based on discharge.

– Interpolating the AEP for each discharge (AEP_i) from the flood frequency curve.
– Interpolating the damage cost (D_i) for each discharge from the discharge-damage curve.
– Calculating the ARC associated with flood damage using Equation 1.

$$\text{ARC} = \sum_{i=1}^{n} \left(AEP_i - AEP_{i+1}\right) \frac{\left(D_i + D_{i+1}\right)}{2} \qquad (1)$$

A summary of the capital, annual and net present costs (2006 AUD) from the options study are included in Table 1. It must be noted that these were initial estimates based on limited design and quantity estimates and do not represent the final project costs.

The options were then compared in a workshop forum using a multi-criteria analysis considering cost, environmental aspects, social/political issues and technical merit.

During the development of these options, it was noted that all the options being considered were generally converging towards the same solution as a result of making modifications to address the same set of risks.

A large proportion of the costs for the "enhanced full fix" option was in mass concrete infill for the "amphitheatre" and "canyon" areas. Clearly, this volume of concrete could be reduced by moving the flip bucket upstream and excavating some of the rock downstream of the adopted flip bucket location. This yielded a modification to the SMEC "intermediate" option.

As part of the concept design process, cost estimates were developed for a range of options with the length of the chute extension ranging from 52 m to 75 m. The latter is effectively the "full fix" location. In consultation with the client, a chute extension length of approximately 61 m was selected for development in the detailed design phase. This decision was based on achieving a balance between minimisation of the cost and maximisation of the protection provided by leaving the granite in place downstream of the dacite.

Table 1. Summary of capital costs and NPC.

Option	"Enhanced anchor & patch"	"Enhanced full fix"
Project cost ($ million)	17.7	29.8
Flood damage ARC ($)	83,900	9,900
O & M annual cost ($)	20,000	20,000
NPC of annual costs ($ million)	1.7–3.4	0.5–1.0
Total NPC ($M)	19.4–21.1	30.3–30.8

3 DETAILED DESIGN

The concept design from the previous phase was used as the basis for the development of the detailed design arrangement. Key features of the design included:

– Filling of the erosion hole ("amphitheatre" and "canyon") with backfill concrete;
– Construction of an anchored reinforced concrete slab in the unlined section;
– Placement of anchored reinforced concrete protection over the downstream cut slope;
– Construction of walls in the spillway chute extension;
– Excavation of a plunge pool below the end of the spillway;
– Anchorage of the existing concrete chute slab to resist uplift pressures;
– Retrofitting of drains in the existing concrete chute slab; and
– Raising of the walls in existing chute to contain the PMF.

An important phase in the design process was the construction of a scale physical model of the spillway as a design verification and refinement tool (Figure 7). The model was constructed at a length scale of 1:60.

Key outcomes of the physical model study included:

– Evaluation and verification of the adequacy of the proposed design for floods ranging from 1:100 AEP (1,050 m³/s) to the PMF (10,500 m³/s);
– Confirmation of the spillway rating curve to ensure the dam was capable of passing the required flood;
– Improvement and refinement of the design through changes to the flip bucket and chute wall convergence;

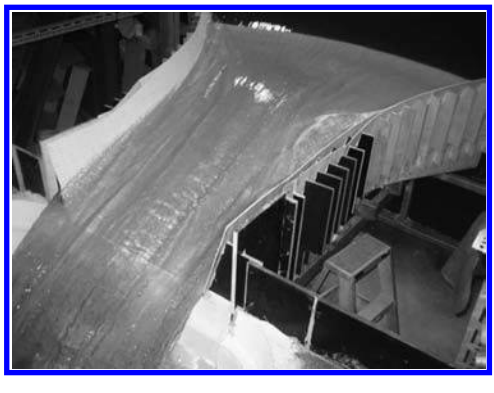

Figure 7. Scale model discharging the PMF.

– Confirmation of spillway wall heights to minimise the risk of overtopping taking into account wave action and other transient effects;
– Incorporation of a pre-excavated plunge pool to improve energy dissipation;
– Monitoring of flow patterns within the plunge pool to minimise potential for erosion by back-cutting; and
– Measurement of downstream tailwater levels and flow velocities in the plunge pool area and the downstream river to assess the risk of erosion of the river channel and possible impacts on adjacent infrastructure including a water main and pump station.

4 PROJECT DELIVERY METHOD

Over the last 10–15 years, south-eastern Australia has been suffering its worst recorded drought in European history. This has seen nearly all major population centres enduring water restrictions. In the early- to mid-2000s, ACTEW as the water supply authority for the ACT, undertook a review of possible water supply projects to augment the supply with the aim of limiting the impacts of future droughts on the residents and business of the ACT. The selected options included a new dam and an inter-basin transfer scheme, both of which are under construction.

The delivery method adopted for these projects was the "Alliance" method, where the owner selects a design and construction partner to form an entity which together with the owner provides the services to deliver the project from the early planning and preliminary design phases through to construction and commissioning. ACTEW elected to include the Googong Dam Spillway project in the Alliance, even though the design had been undertaken prior to the formation of the Alliance. GHD had been engaged to undertake the detailed design and preparation of the contract documentation for the upgrade. Upon the formation of the BWA the design was passed on to the Alliance to undertake the construction. This allowed the construction partner within the BWA to mobilise relatively early in the overall Alliance program, which would not have been possible on the other projects as their design was not as advanced.

5 CONSTRUCTION PROGRESS

Construction of the remedial works commenced in January 2009 and is due for completion in late 2010. Figure 8 shows an overview of the site from late May 2010. The following points summarise the construction progress to late May 2010:

Figure 8. Overview photo showing construction progress to end of May 2010.

– Backfill concrete placement complete in the "amphitheatre";
– Rock excavation, foundation cleanup and dental concrete placement complete in chute extension;
– All concrete placement complete for raising of the existing spillway walls;
– All blasting complete for rock excavation downstream of the flip bucket and in the plunge pool;
– Concrete placement over 50% complete for the gravity spillway walls in the chute extension;
– Installation of anchorage for the slab in the chute extension nearing completion; and
– Concrete placement for the reinforced concrete apron below the flip bucket over 50% complete.

Some of the key challenges encountered and overcome throughout the construction of the spillway remedial works are discussed below.

6 CONSTRUCTION CHALLENGES

6.1 *General*

The challenges that arose during the construction of the works can be categorised into four broad areas:

– Flood risk related to working in a live spillway with limited or no control of the reservoir level;
– Constructability/program issues where potentially more efficient construction methods were identified which required a review of the design;
– Health and safety issues where undue risk was placed on the construction workers requiring review of the design and/or construction methodology to mitigate the risk; and

– Environmental risk associated with impacts on the natural environment and the proximity to the water supply reservoir and watercourses.

6.2 *Flood risk*

A major project risk identified during the design phase was that of working on a spillway for an active reservoir where there was little control over water levels in the storage. A strategy was required to minimise the risk to the project in terms of cost, program and prevention of further damage to the existing spillway components and damage to the completed or partially completed upgrade works.

At the outset, a range of diversion strategies were developed which generally focussed on protecting a part of the spillway while the remedial works were undertaken in that area. Each strategy was assessed through modelling the reservoir behaviour using 136 years of historical rainfall and inflow records and the proposed construction program. The historical record was broken up into 68 "independent" sets of two-year records representing the period from the date of the original assessment to the end of construction. The assessment methodology was as follows:

– The first part of each of the 68 records was used to simulate the operation of the reservoir from a known starting water level at the time of the assessment through to the start of the construction period resulting in a water level exceedance curve for the start of construction;
– A range of water levels for the start of the construction period were selected from the curve;
– Each of these starting water levels was then used as the starting water level to model the reservoir behaviour during construction (using the remaining part of each of the 68 records) and its effect on the construction activities;
– For each of the 68 runs, a count was kept for a range of parameters such as the number of spill events and the number of days on which the reservoir level exceeded a pre-determined threshold where construction work would be delayed;
– For each starting water level, percent exceedance relationships were developed from the data collated from the 68 runs for each of the parameters being considered (ie risk cost and number of spill events and days delay);
– These percent exceedance relationships were then integrated to give the best estimate for each parameter conditional on the starting water level for the set of runs; and
– This data was then integrated with the starting water level exceedance curve to give the total risk for that parameter for the diversion arrangement under consideration.

Diversion arrangements originally considered included a "one third-two third split" of the spillway (ie work on one-third of the spillway while diverting flood flows down the remainder, then change over), vice versa and a "half-half split" of the spillway (ie work on half while diverting flood flows down the remainder, then change over). These options were compared against a "no diversion" case. The diversion structures considered ranged in height up to 2 m. A summary of the assessment results for a 1.5 m high diversion structure is included in Table 2.

As the summary indicates, a significant benefit was achieved in terms of potential savings in both time and cost when a diversion structure was installed. On this basis, a decision was made to incorporate a diversion structure to pass minor floods. This would allow work to continue, particularly during the tail end of these events, once the storm event had passed. The structure was designed to fail with less than 0.5 m depth of overtopping to ensure that the full spillway would be available for the passage of large floods.

Throughout construction, the project has benefited from the ongoing dry conditions and low storage levels, and to date no significant inflow events have occurred and the reservoir has not risen significantly from pre-construction levels.

6.3 *Constructability/program issues*

6.3.1 *Thermal stresses in large concrete pours*

The concrete construction specification was developed to minimise cracking, produce a durable concrete and minimise the introduction of destabilising pressures to the walls and slabs during flood events. This required tight restrictions on the concrete placement temperature, peak temperature rise and the size and sequence of pours.

These restrictions imposed significant constraints on the construction of the extended spillway chute walls, which are mass concrete gravity structures, up to 17 m high and 10 m thick at the base. This led to a comprehensive review of the design and construction methodology in an attempt to identify the most efficient means of constructing the walls whilst still meeting the design intent.

Table 2. Comparison of results for "half-half split" diversion and no diversion cases.

Parameter	"Half-half split"	No diversion
Risk cost (AUD million)	1.38	2.41
No of spill events	≤1	3
Delay time (days)	88	136

It should be noted that heat generation in large concrete pours is a major issue in the civil construction industry in Australia. The cement industry has generally been influenced by its major customers, the building industry, who need high early strength cements. Therefore, cements are often finely ground and it is difficult to get more coarsely-ground cements that are better suited to mass concrete applications.

The original design and specification of the gravity walls called for a 20 MPa concrete core (to minimise heat generation) with a 32 MPa reinforced skin (to meet code durability requirements for resisting freeze/thaw action). The skin concrete was to be poured integrally with the concrete core. The maximum temperature of the concrete at placement was set at 20°C and the peak temperature was limited to 50°C. The maximum allowable pour height was 1.5 m and a subsequent pour had to be placed within 3–7 days of completing the previous pour.

The constructor identified that this was going to place a number restrictions on the program, and the designer and the constructor set about reviewing the design requirements and specified limits.

Early in the project a number of mass concrete pours were placed which did not have the durability requirements. These pours were extensively monitored to determine the heat generated and the expected temperature rise. Insulated instrumented mass concrete blocks were also poured to gain further knowledge on the heat generation likely in mass concrete pours.

This data was used to model the stress development in the walls resulting from the heat generation and dissipation within the structure. This extensive modelling considered different construction sequences pour heights, placement temperatures, pre- and post-cooling options and cement cont-ents for both core and skin concrete. The modelling indicated that the original design was valid; however some minor relaxation of the limits was permissible.

The final design to be constructed was the original design cross section with a 15 MPa core concrete and 32 MPa reinforced concrete skin. The maximum placing temperature was retained at 20°C however rules where put in place to allow some loads to be placed as high as 23°C provided the overall average temperature of placed concrete met the 20°C limit. The peak temperature was retained at 50°C as the modelling showed cracking tended to increase when the temperature exceeded this limit. The maximum pour height was increased to 2.1 m and the maximum interval between subsequent pours was increased to 9 days.

The revised specification requirements still placed a considerable restriction on the construction program. However the constructor has generally met these limits, and on those occasions where the limits have not been met there has not been a significant impact on quality. Cracking observed in the mass concrete walls to date has generally been within the limits anticipated at the design stage.

6.3.2 *Impacts of blast vibrations*

To meet program it was necessary to progress the works on a number of fronts. This required excavation, including blasting of rock, to be undertaken in the vicinity of concrete works and grout-encapsulated dowels. Blast energies had to be limited to protect the long-term integrity of the concrete and the grout. This was controlled by limiting Peak Particle Velocities (PPV) at the location of the new works, depending on the age of the concrete. Note that PPV limits were also applied at existing infrastructure, however these limits did not control. A further restriction was the proximity of a blast to any new works which required careful coordination of the sequencing of blasts and concrete pours to ensure the limits were met.

During the works, it became evident that to minimise disruption to the program not all the limits could be met for each blast, particularly on the grouted dowels forming the rock support. A significant concern with these dowels was the potential for cracking of the grout encapsulation leading to increased corrosion of the steel bar.

A series of test anchors and other trials were prepared to provide some confidence that a relaxation on the blasting limits was warranted. The test anchors were load tested before and after the blast and did not show any reduction in capacity. Water tests were also undertaken on open holes adjacent to the blast to assess whether the blast vibrations were opening up defects within the rock mass. These tests did show some a minor but insignificant increase in water takes. It was concluded that the PPV on the grouted anchor bars could then be increased.

6.4 *Health and safety issues*

Australian industrial safety legislation places the onus on the designer and constructor to provide a safe work place throughout the life-cycle of a project. This legislation is as onerous as any in the world. This can impact on the design as at times compromises are required between achieving the required design intent and the safety of those undertaking the construction activities.

This is highlighted with the construction of the spillway chute extension slab. The design had required a "checkerboard" pattern of pouring the slab to maximise the shrinkage on one slab before the adjacent slabs were poured, hence minimising the potential opening of the joints. These slabs

are secured to the foundation by passive dowels installed prior to the slab being poured.

As the works progressed, it became obvious that access to the sloping spillway chute by the drill rig used to install the anchors was not as safe as required and alternate means of providing access for the drill rig onto the chute had to be found. This impacted on the program and as a result the sequence of placing the slabs had to be modified to limit the impact on the overall construction program.

A review of the likely shrinkage was undertaken to determine if an alternative pattern could be used without compromising the original design intent of minimising the joint opening. Ultimately, it was not possible to fully achieve, however, with some compromise on the acceptable shrinkage and the removal of a number of joints, a sequence was developed which for all practical purposes met the design intent.

6.5 *Environmental risk*

Googong Dam and the area surrounding the dam are located within the Googong Foreshores, a reserve which is managed as a water supply catchment, a public recreation area and as a wildlife refuge for many plant and animal species including a number which are threatened.

The project faced considerable risk in managing potential environmental impacts associated with pollution of waterways under multiple jurisdictions—the Googong Reservoir managed by the ACT government as well as the Queanbey an River regulated by the NSW government. Negotiations with both ACT and NSW governments and the owner over the management of environmental risks adjacent to these respective waterways led to the development of a Construction Environment Management Plan that could be practically implemented throughout the construction phase of the project. Implementing erosion and sediment control plans that enabled the capture of turbid water within sediment basins and diversion of clean water flows where practical was critical to meeting stringent environmental performance criteria stipulated by regulatory agencies.

Minimising impacts to local biodiversity, notably an endangered ecological community (White Box, Yellow Box Box, Blakely's Red Gum Woodland), was another key environmental risk that warranted careful management by the site team. This entailed delineation of clearing areas, identification by botanists of hollow bearing trees that should be retained on site for fauna habitat and the preparation of a biodiversity offsets management plan to be implemented within the Googong Foreshores Management Area.

7 SUMMARY AND CONCLUSIONS

The spillway at Googong Dam has undergone significant erosion of the unlined section of the spillway subject to relatively infrequent flood events. Furthermore, the revised PMF outflow could not be contained by the existing chute walls. An assessment of the potential erosion indicated that without some strengthening works and the raising of the chute walls the risk of the erosion extending back to the reservoir was unacceptable. ACTEW therefore elected to undertake the upgrade works for the spillway.

The works have been undertaken in a timely and cost effective manner, to bring this important dam up to the level of safety required for an extreme consequence category dam. The works undertaken will allow the spillway to safely pass the PMF.

A number of challenges were encountered as construction progressed, which had the potential to compromise the program and/or quality of the works. However, the Alliance delivery method put in place by ACTEW permitted a co-operative approach between owner, designer and constructor allowing solutions to the issues to be identified and developed whilst minimising program delays and compromise to the design intent.

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Recrecimiento del vertedero de la presa de Urra: Optimizar la capacidad de embalse preservando el medio ambiente en Colombia

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ABSTRACT: URRA ESP SA has just equipped the spillway of its dam with a mixed system of fusegates and flap-valves to enhance the normal pool of 2 m.

URRA I DAM is designed for multipurpose flood management and hydropower production in the Sinú river in northern Colombia. The embankment dam of maximum height of 73 m and is equipped with a 182 m free overspilling spillway that culminates assistant 8,5 m below the crest of the dam.

URRA SA ESP sought to enhance the storage level of its 1740 hm^3 dam to improve flood mitigation whilst increasing its production of electricity without compromising the safety level of against large floods.

The project of increase is exemplary; both in its integration into the environment and capacity for additional production are offered by optimizing the use of tidal areas, owned by the same company URRA ESP SA.

Beyond the technical challenge, the article presents the project, the conditions of installation and operation, and integration of the various environmental parameters studied (populations upstream and downstream, biotopes, water quality, risk of vandalism ...).

The first reference to Hydroplus in South America illustrates the various interests of the system and proposes an alternative for the optimization of existing Dam and their production capacity.

RESUMEN: URRA ESP SA acaba de equipar su vertedero de un sistema mixto de compuertas fusibles y de válvulas de tipo clapeta para realzar la cresta de 2 m.

La represa de URRA I es una represa multipropósito diseñada para la gestión de crecidas y la producción de energía hidroeléctrica en el río Sinú en el norte de Colombia. La presa de terraplén de altura máxima de 73 m y está equipada con un vertedero libre de 182 m que culmina 8,5 m por debajo de la cresta de la presa.

URRA ESP SA buscaba solución para realzar el nivel de almacenamiento de la presa URRA I de capacidad de 1740 hm^3 para mejorar la atenuación de crecida y aumentar su producción de electricidad sin perjudicar al nivel de seguridad de su presa frente a crecidas mayores.

El recrecimiento de esta obra es ejemplar, tanto en su integración en el medio ambiente como por la capacidad de producción adicional que ofrece la optimización del uso de las zonas de mareas, cuyo dueño es la misma empresa Urra ESP.

Más allá del reto técnico, el artículo presentara el proyecto, las condiciones de instalación y de funcionamiento, y su integración ambiental con los diversos parámetros que se estudiaron (poblaciones aguas arriba y aguas abajo, biotopos, calidad del agua, riesgos de vandalismo …).

La primera referencia de Hydroplus en América del Sur permite ilustrar los diversos intereses del sistema y propone una alternativa para la optimización de las obras existentes y de su capacidad de producción.

1 INTRODUCCIÓN

El valle del río Sinú, en Colombia, siempre fue sometido a crecidas recurrentes muy importantes. Las poblaciones indígenas habían construido hace siglos un sistema muy complejo de canales y desviaciones que permitían alternar entre actividades lacustres y agricultura en las mismas áreas dos veces por año, aprovechando las variaciones del caudal del río Sinú. De estos canales, quedan ahora huellas geológicas que se pueden observar desde el aire. Varios museos documentan esta ingeniería agrícola integrada, entre ellos el museo del Oro de Bogotá, Colombia. Desde el principio del siglo XX, el desarrollo económico vio instalarse poblaciones, ciudades y campos de agricultura y ganado que son muy vulnerables a las crecientes. La gestión de las crecidas y la producción de energía son dos parámetros muy importantes en esta zona del norte de Colombia. La presa Urra I tiene el propósito de responder a estas preocupaciones mayores.

La presa Urra I es operada desde el año 2000 con la finalidad de abastecer la central hidroeléctrica epónima con una potencia instalada de 340 MW, mediante cuatro unidades tipo Francis de 85 MW cada una, y al mismo tiempo de laminar las crecidas del río. La represa de Urra I con 7400 ha, es la más grande en Colombia, tendiendo un dique principal de 660 m de ancho con una altura máxima de 73 m. La presa permite almacenar hasta 1740 Mm^3 de los cuales 1200 Mm^3 representan el volumen útil que permite generar hasta 1300 GWh/año.

La empresa de servicio público Urra SA ESP opera y administra en forma sostenible la central Hidroeléctrica Urra I y comercializa competitivamente la energía eléctrica. Sus operaciones son desarrolladas con base en criterios de eficiencia, calidad, idoneidad, confiabilidad, mínimo costo, sostenibilidad ambiental y alto sentido de responsabilidad social.

Desde su diseño fue previsto que la presa pudiese responder a dos criterios de crecida mayores mediante un vertedero de labio fijo de 182 m de ancho con un borde libre de 8,5 m. El vertedero permitía evacuar la crecida de periodo de retorno de 10 000 años, correspondiendo a un doble pico de 4500 m^3/s en 300 horas, bajo una lamina vertiente de 3,5 m por encima de la cresta del vertedero, y evacuar la crecida máxima probable (PMF), correspondiendo a un pico de 14 900 m^3/s en 100 horas bajo una lamina vertiente de 7,40 m.

También se integró en las reglas de explotación limitar lo mas posible el impacto de las crecidas del río Sinú en las poblaciones aguas abajo. Para cumplir con este criterio, el vertimiento acumulado del vertedero más el caudal evacuado producto de la generación, nunca debe de sobrepasar los 700 m^3/s, caudal este el máximo que soporta el cauce sin desborde del río Sinú. Esta regla impone que, en la medida que se va incrementando el caudal vertido por el vertedero, se deben ir sacando de operación las unidades de generación secuencialmente. Expresado de otra manera, $Qv + Qg \leq 700\ M^3/S$.

Figura 1. Vista desde aguas arriba del vertedero durante la instalación de las compuertas fusibles.

2 DEFINICIÓN DEL SISTEMA DE RECRECIMIENTO DEL VERTEDERO

Desde los inicios de operación de la central hidroeléctrica se había empezado a indagar de qué manera se podría incrementar la capacidad de almacenamiento del embalse sin comprometer la seguridad estructural y operacional de la planta. A principios del año 2007, se consolidó esta idea y se inició la gestión ante el Ministerio del Medio Ambiente.

En los diez primeros años de operación, el vertedero funcionó en dos temporadas y evacuó hasta 900 m^3/s. En estas ocasiones se pudo observar el impacto de las crecidas en las poblaciones aguas abajo, y fue necesario parar la operación de la central para limitar el caudal evacuado. La temporada de invierno del mes de **junio de 2007 originó una condición de rebose con caudales vertidos de 700 m^3/s que implicó tener que apagar la planta siguiendo la reglas de operación La pérdida de explotación que se generó se pudo estimar en 2,5 millones USD.**

Este evento se puede asociar con un periodo de retorno de aproximamente 20 años, y corresponde a un aumento del nivel del embalse de 1,40 m por encima del labio del vertedero. Se estimó que una capacidad de almacenamiento adicional hubiese podido limitar de manera importante el impacto de tal tipo de crecida.

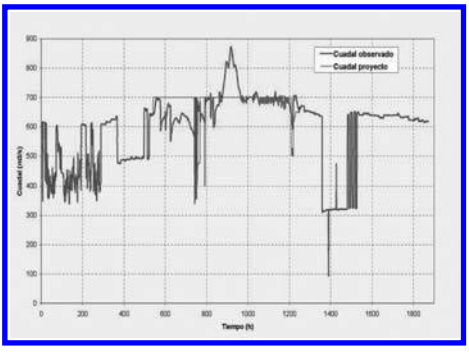

Figura 2. Comparación de descarga caudales aguas abajo durante la crecida de 2007—regulación de caudal autorizada con el proyecto de recrecimiento—los caudales por encima de 700 m^3/s provocan inundaciones graves aguas abajo.

Al sobrepasar el caudal de 700 m^3/s, la crecida provocó inundaciones mayores aguas abajo, las cuales afectaron a mas de 55 000 habitantes y generaron daños cuyos costos se estimaron en 5 millones de USD.

Este ejemplo ilustra un interés ambiental importante para las poblaciones aguas abajo. Igualmente vale la pena destacar que el proyecto fue objeto de un estudio específico de impacto medio ambiental que se presentó al Ministerio de Medio Ambiente. La valoración de los impactos llegó a la conclusión de que no se necesitaba modificar la licencia ambiental. Se destacaron los puntos siguientes:

- Pérdida de vegetación en la franja de oscilación del embalse: la modificación del proyecto solo inundará con mayor frecuencia un área que fue prevista en el proyecto inicial que se pudiera inundar esporádicamente, esta zona es propiedad de URRA ESP SA y está prevista para inundaciones temporales en la licencia ambiental; se conservara parcialmente vegetación natural resistente a las oscilaciones del embalse. El impacto fue calificado de bajo.
- Afectación de la calidad del agua en el embalse: el incremento de las zonas inundadas parcialmente, y por menos de un mes por año corresponde a 4.6% de la superficie inundada. Los modelos de modificación de calidad de agua por inundación de la biomasa indican un impacto calificado bajo.
- Estabilidad de la presa y diques auxiliares por saturación de agua: Estando el nivel de explotación dentro de las elevaciones previstas en el diseño original, no se identifico ningún impacto adicional. Se hará un monitoreo más detallado en estas estructuras por la permanencia del nivel. El impacto es irrelevante.
- Impactos sociales aguas arriba: Solo se inundan algunas construcciones temporales que fueron construidas sin autorización en zonas de seguridad cuya propiedad es de URRA SA ESP. En proceso de reubicación estas familias, previa advertencia antes de empezar llegar a cota de inundación. El impacto es calificado como bajo.
- Impactos Ambientales y Sociales Aguas Abajo de la Presa durante la Operación. Mejor control de las inundaciones, mejorándose la calidad de vida de los habitantes rivereños. Este impacto positivo tiene un alto contenido social.
- Aspecto financiero: La recuperación de los costos se estima en cuatro años, lográndose **un "nuevo embalse" con un costo 13 veces menor** si se hubiese construido uno de igual capacidad con métodos constructivos convencionales.

El interés ambiental y económico convirtió la solución en una realidad.

3 ALTERNATIVAS

Varias alternativas fueron consideradas para la definición del sistema de recrecimiento del vertedero, integrando criterios de impacto ambiental, de optimización de recursos y de limitación de la obra civil.

Entre estas se destacaron la utilización de compuertas inflables y de compuertas fusibles. Tras una valoración de ventajas comparativas hecha por el dueño de la central con la asesoria técnica de la empresa Gómez, Cajiao S.A., se destacaron varias ventajas específicas que apuntaron a seleccionar la implementación de compuertas fusibles.

Las compuertas fusibles presentan las ventajes siguientes:

- El sistema de compuerta fusible tiene un alto nivel de confiabilidad, por ser totalmente independiente de cualquier operación humana, y de todo sistema de control y de energía.
- Los materiales de construcción permiten una alta durabilidad en un entorno severo por condiciones climáticas. No hay partes sueltas y sometidas a cargas variables durante la operación.
- La inversión inicial y el costo de mantenimiento son muy reducidos para esta solución. El mantenimiento se limita a una verificación visual semestral y a un tratamiento de pintura esporádico.
- El diseño, el suministro y la instalación se pueden adaptar al tiempo de la obra. En este proyecto era necesario realizar estas operaciones en menos de un año.
- La estabilidad de las compuertas fusibles puede ser asegurada contra situaciones críticas y varios mal-funcionamientos desde su diseño, integrando incluso las posibles acciones de vandalismo.

- La obra civil necesaria para la instalación de las compuertas fusibles es muy reducida y se limita al anclaje de topes de apoyo aguas abajo.

El sistema que fue finalmente seleccionado permite entre otros objetivos cumplir con los requisitos de laminación de crecidas intermedias, facilitando evacuar todas las crecidas sin presentarse ningún volcamiento de diques, hasta la que la creciente de los 100 años se de.

Unas compuertas fusibles con capacidad de vertimiento optimizado fueron diseñadas para este proyecto.

Varias alternativas de altura de recrecimiento se consideraron para optimizar el almacenamiento.

Tras el estudio de impacto ambiental, se seleccionó una solución que cumplía con el nivel máximo que alcanzaría el embalse en presencia de la avenida máxima probable (PMF) que corresponde a la cota 135.90, pero que también cumplía con la condición de evacuar la crecida intermedia con periodo de retorno de 10 000 años por debajo de la cota 132,00 msnm, la cual esta licenciada (es decir solamente 3,5 m, de los 8,5 m de borde libre, por encima del labio del vertedero actual).

Esta condición creó problema importante sobre la definición de los parámetros de las compuertas fusibles, lo cual obligo que el sistema clásico de compuertas fusibles fuese completado mediante la instalación de dos compuertas tipo clapeta de 5,5 m de altura, accionadas mediante pistones y cadenas, que fueron instaladas en los dos vanos laterales del vertedero, cada una de 14 metros. Entre otras ventajas, este sistema permitió también limitar las obras de demolición a 2 de los 13 vanos del vertedero.

El sistema mixto de compuertas fusibles y de compuertas clapeta permite a su vez aumentar el nivel de almacenamiento en 2,0 m mientras se conserva la capacidad inicial de desagüe del vertedero para las crecidas mayores.

Se presenta a continuación un esquema de varias alternativas consideradas para la realización del proyecto.

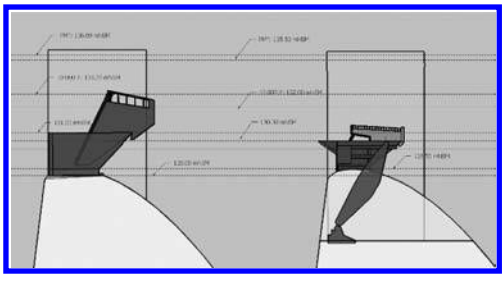

Figura 3. Comparación de alternativas de recrecimiento con demolición en todos los vanos (1) o con demolición limitada a 2 de los 13 vanos (2).

4 FUNCIONAMIENTO

Para cumplir con las reglas de explotación y los requisitos de medio ambiente se definió el sistema mixto descrito de compuertas fusibles y de clapetas. Las clapetas funcionan con un autómata local que podrá luego ser comandado mediante un sistema de Centralog que instalaría Urra. El funcionamiento define una apertura progresiva asociada al nivel de agua en el embalse, de manera tal que las clapetas **no** se abran a menos que el agua vierta por encima de las compuertas fusibles. En el caso de no poder usar el sistema de control, las clapetas poseen un flotador que permite una apertura mediante un sistema de by-pass hidráulico.

El funcionamiento intrínseco de las compuertas fusibles y los parámetros de diseño permitiendo enfrentar todos los casos adversos han sido descritos en varios artículos. Las referencias se encuentran en la parte 'publicaciones' de la pagina internet de Hydroplus http://www.hydroplus.com/es.

Para este proyecto se diseñaron compuertas fusibles con geometría optimizada para lograr un alto coeficiente de descarga y la facilidad de poderse instalar directamente en la gola del vertedero sin tener que hacer obras de demolición del hormigón importantes. Veintidós compuertas fusibles de 7 m de ancho por 2 m de altura fueron-instaladas en 11 de los 13 vanos del vertedero. Las compuertas fueron diseñadas para limitar el impacto de los cuerpos flotantes dejando una luz entre pozos de admisión de 10 metros. Las compuertas fusibles están diseñadas para que todas ellas se queden en su sitio hasta que se presente la avenida de 100 años, que corresponde a un caudal saliente del orden de 1700 m^3/s, y para que la apertura se haga de manera secuencial hasta que el nivel aguas arriba llegue a la cota 132.00 msnm para la crecida con periodo de retorno de 10 000 años.

Las compuertas fusibles funcionan de manera independiente de todo tipo de control y son

Figura 4. Modelo reducido de compuerta fusible en fase de prueba.

activadas por admisión de agua en el pozo superior en caso de crecidas mayores. Antes de esta activación, permiten un vertimiento de más de 1,75 m por encima de su cresta. Cada una de las compuertas fusibles funciona de manera independiente y secuencial por tener pozos de admisión a diferentes alturas. Cada compuerta fusible tiene una estabilidad propia afinada mediante lastres para lograr los objetivos de fortaleza en todos los casos adversos, como vandalismo, deterioración de los sistemas de juntas, impactos de cuerpos flotantes y olas.

Las compuertas fusibles fueron ensayadas con el cliente en el laboratorio de Hydroplus en Francia.

5 VENTAJAS PARA ALMACENAMIENTO, LAMINACIÓN DE CRECIDAS Y CAPACIDAD DE VERTIMIENTO

El interés de la solución de propuesta puede presentarse bajo varios enfoques:

- Laminación de crecidas:
  - Para eventos comunes: el aumento de la capacidad del embalse permite incrementar la laminación de la crecida. El caudal de vertimiento alcanza picos menores y limita las inundaciones aguas abajo.
  - Para eventos mayores: una apertura secuencial de las compuertas fusibles del vertedero permite un aumento progresivo de la capacidad de alivio. No hay aumentos rápidos de caudales aguas abajo y no se provocan inundaciones adicionales.
  - Para todos lo eventos de crecidas con periodo de retorno inferior a 100 años, es posible almacenar 150 Mm^3 adicionales sin vertimiento. Para eventos con periodo de retorno mas elevado que el de 100 años, se conserva la capacidad de almacenamiento original, y es posible volver a instalar compuertas fusibles en corto plazo. El cliente teniendo todos los planos de fabricación y la patente para las compuertas fusibles para su vertedero, puede hacerlo a su discreción cuando quiera.
- Almacenamiento: el realce permite incrementar la capacidad en 150 Mm^3 sin obras mayores.
  - El aumento de la cota de rebose permite incrementar en promedio la generación de energía, en 122 GWH año, lo que significa un incremento del 9.4% en la producción media anual.
  - La capacidad extra incrementa la producción de la central hidroeléctrica. Para comparición, el volumen de 150 hm^3 corresponde a una generación de 7 días a plena carga de la central de 340 MW.
  - Durante los eventos de crecida que provocaban desagües por el vertedero, el operador tenía que parar la producción eléctrica para limitar el caudal del río aguas abajo. Al aumentar la capacidad del embalse, la probabilidad de ocurrencia de tales eventos se disminuye a 20 años como mínimo.
  - Almacenar agua durante los periodos de aportes importantes permite también aumentar la capacidad de producción de manera inter-estacional dando más capacidad de producción para los periodos de aportes bajos (durante los cuales el precio de venta de la energía es más alto).

Se pudo estimar que el recrecimiento sustituye la instalación de un central hidroeléctrica de 29 MW o una térmica a carbón de 20 MW.

- Capacidad de vertimiento: La solución implementada actúa como un vertedero libre con coeficientes de descarga evalucionando según el caudal de las crecidas. Se pueden definir cuatro rangos de funcionamiento:
  - Para un nivel de embalse a la cota de la cresta de las compuertas fusibles, se presenta una pantalla estanca que aumenta el almacenamiento.
  - Para un nivel de embalse entre las cotas 130.50 y 131,74 msnm, las compuertas fusibles actúan como un vertedero de borde libre con un alto coeficiente de descarga. La clapetas se abren progresivamente hasta llegar a un caudal des descarga de 1 700 m^3/s.
  - Para un nivel de embalse entre las cotas 131.74 y 132,00 msnm, se voltean de manera consecutiva las compuertas fusibles, *permitiendo aumentar progresivamente* la capacidad de descarga hasta poder evacuar el caudal de 3500 m^3/s a la cota 132.00 msnm.
  - Para un nivel de embalse entre las cotas 132,00 y 135,53 msnm, el vertedero actúa como un aliviadero de borde libre con capacidad de descarga hasta un caudal de 8500 m^3/s.

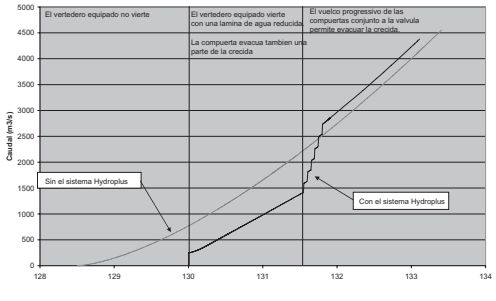

Figura 5. Comparación de capacidad de descarga del vertedero original y del vertedero equipado.

6 TRABAJOS DE OBRA CIVIL Y DE INSTALACION

La realización del proyecto en el sitio de la obra permitió llevar varias innovaciones y operaciones de ingeniería precursoras en Colombia.

La Sociedad Colombia de Ingenieros otorgó a URRA ESP SA un premio en la categoría de Mención de Honor por la tecnología de punta aplicada en la obra. El Presidente de la Republica de Colombia entregó el premio el 28 de mayo de 2010.

6.1 *Demolición con hilo de diamante*

Por requisito del cliente, se había definido limitar el trabajo de obra civil en el mínimo número de vanos. El diseño había entonces definido hacer demoliciones en los dos vanos laterales, con un volumen total de 1058 m^3. Durante la obra se pudo comprobar que la resistencia del hormigón del vertedero era muy elevada, llegando a más de 5000 PSI en los ensayos destructivos que se realizaron. Por otra parte la tarea de demolición no podía debilitar las pilas del puente que corre por encima del vertedero, ni tampoco afectar la galería de drenaje situada bajo el perfil ogee. Era entonces imposible utilizar técnicas destructivas tradicionalmente utilizadas para demolición tales como explosivos, quebrantadores o picadora. Las cantidades de demolición tampoco permitían utilizar técnicas locales que generan grandes cantidades de escobros e implican tiempos largos de operación. Fue seleccionada entonces una técnica de demolición mediante hilos de diamante utilizando una maquinaria con sistema integrado de almacenamiento de hilo. La demolición se realizó bajo un control topográfico muy preciso pues era imprescindible seguir fielmente guías de obra civil. La aplicación de demolición con hilo de diamante para este vertedero fue la primera realizada en Colombia, pero también constituyó un gran desafío debido a las condiciones muy difíciles de acceso y a la precisión del corte que se requería.

Figura 6. Bloque de hormigón cortado durante su extracción.

6.2 *Fabricación Colombiana de módulos ensamblados en sitio*

Las clapetas y las compuertas fusibles fueron realizadas en talleres de Colombia. Tras consulta con proveedores locales, se decidió fabricar los elementos en talleres de construcción metálica en Bogotá para poder cumplir con especificaciones geométrica precisas. La empresa Elementos Metálicos bajo un control de calidad muy alto fabricó más de 300 toneladas de metal, constituido principalmente por acero ASTM A32, pero también por materiales especiales para piezas de manejo de las clapetas. Por razones de transporte, y con el fin de facilitar las operaciones de montaje, los elementos se construyeron en piezas que se ensamblaban en el sitio de la obra. Cada compuerta fusible está constituida por 4 elementos y las clapetas por 11 elementos cada una.

6.3 *Instalación bajo alto criterios de seguridad, de calidad y de control del medio ambiente*

Bajo la supervisión y la asistencia técnica de Hydroplus, la empresa EMEC Ltda. realizó la instalación tanto de todos los elementos metálicos

Figura 7. Compuerta fusible ensamblada en sitio.

Figura 8. Instalación de las compuertas fusibles desde el puente.

como del material de control y manejo importado sin contemplar accidente alguno. Todas las operaciones fueron llevadas con seguimiento topográfico continuo, y el control de calidad con pruebas y registros permanentes. La instalación de las compuertas fusibles se realizó en menos de un mes mientras que las clapetas necesitaron tres meses para su instalación.

7 CONCLUSIONES

El proyecto de recrecimiento del vertedero de Urra ilustra los intereses medio ambientales y económicos del recrecimiento basado en tres funciones fundamentales del proyecto: el control de las inundaciones, el almacenamiento de agua y la producción de energía hidroeléctrica.

Esta obra es la primera obra en su género en Latinoamérica. Es muy importante destacar que mediante innovación y los aportes técnicos en el proyecto, el recrecimiento del vertedero permite mejorar el control de inundaciones sin impactos ambientales relevantes, y mejorar al mismo tiempo la generación de electricidad.

Este proyecto constituye un referente en el sector eléctrico permitiendo aumentar la producción de energía con costos reducidos. Se calculo que el costo era por lo menos 13 veces menos elevado que mediante la instalación de nueva presa.

La integración ambiental y social del proyecto de recrecimiento sugieran a estudiar la optimización asegurada de la presas con este tipo de procesos.

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Refurbishment of Klingenberg dam

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ABSTRACT: The Klingenberg dam is located south-east of Dresden (Saxony, Germany). It is a masonry gravity dam with a curved axis and a sealing of Preolite coating behind the concrete upstream face. The dam was built from 1908 to 1914 and it is currently used for drinking water supply, flood control and hydro-electric power generation.

After almost 100 years of operation, there are signs of structural damage on the dam, the bottom outlets, intake system and other components of the dam. In particular, the spillway was seriously damaged during an extreme flood event in 2002. During the 2002 flood event, the discharge over the spillway was 75% higher than its design capacity.

After the 2002 event, intense rehabilitation and repair works have been conducted with a view to bringing the dam to the recommended standards. The main works at the dam have been:

- building of a new upper dam,
- construction of a 3,3 km long tunnel from above the pre-dam around the whole reservoir as an additional spillway and a facility to improve water quality during floods and for securing adequate drinking water supply for the city of Dresden during the rehabilitation of the main dam (empty reservoir),
- increase of the spillway capacity to cope with the new design flood,
- construction of a new inspection gallery into the dam,
- refurbishment of the dam (sealing, monitoring instrumentation),
- construction of new hydraulic elements (outlets, intake system, stilling basin).

This article discusses the most important aspects of the refurbishment works, the role of the dam for flood protection and water supply of the Saxon capital Dresden.

1 INTRODUCTION

1.1 *Stewardship of Saxony's waters—function of the State Reservoir Administration of Saxony*

The State Reservoir Administration of Saxony (Saxony, Germany) was founded in 1992 as the first public enterprise in the Free State of Saxony. It is a part of the Saxon State Ministry for Environment and Agriculture and carries out numerous sovereign duties. These include raw water supply, watercourse maintenance and flood protection. The duties of the State Reservoir Administration of Saxony are defined in the Saxon Water Ordinance.

Almost half of Saxony's drinking water comes from reservoirs. The State Reservoir Administration of Saxony operates 23 drinking water dams with an overall storage capacity of 236 million cubic meters. It is responsible for the planning, construction and maintenance of all state dams and providing water for drinking and processing. Many of Saxony's dams are linked together through a network system. If one reservoir experiences a shortage, the raw water can be supplied to the waterworks through a system of pipes, galleries, rivers and streams, ditches and ponds from another reservoir. Thus the water supply can be balanced during prolonged dry periods.

The 33 process water dams are used for low water aggradation and to provide water for industrial use. Many of the process water reservoirs are also used for tourism. There are also more than 80 additional reservoirs, like pre-dams, retarding basins and others.

An essential task of the State Reservoir Administration of Saxony is the constant upkeeping of the main rivers, creeks and other watercourses and transboundary waters. This includes in particular the development and maintenance of the banks and embankments, state-owned flood protection dykes and hydro engineering facilities.

The State Reservoir Administration of Saxony is responsible for:

- 137 dams including pre-dams and basins with about 600 million cubic meters of overall storage capacity,
- 4 raw water transfer lines for drinking water purposes,
- 1 process water transfer line system,
- 1 man-made ditch and water gallery system,
- 186 weirs,
- around 400 additional water management facilities of the Free State of Saxony such as water drawing machines, pumping stations and water gauges,
- around 3,000 km of first order rivers and streams,
- 300 km of transboundary waters flowing through Germany, Poland and the Czech Republic,
- around 650 km of flood protection dykes.

1.2 *Klingenberg dam*

Klingenberg dam was built in the period 1908–1914 following a multi-purpose project from the Royal Saxon Waterworks Directorate to reduce the risk of flooding downstream, maintain a minimum water level downstream and to provide drinking water to the Weisseritz area.

Klingenberg dam is a 40 m high gravity masonry dam with arched axis. The dam was made water tight by applying a preolit coating behind a protective casing on the upstream face. It was built following the Intze-system and has no underground impermeability.

Together with the Lehnmühle dam, built between 1926 and 1931, the Klingenberg dam is part of the Klingenberg/Lehnmühle integrated reservoir operation system. This dam system nowadays is not only responsible for the flood protection of the downstream area, but also guaranties the necessary high quality raw water for the drinking water supply of Dresden, the state capital of Saxony, and the Freital area.

The Klingenberg and Lehnmühle dams have a normal flood storage room of 2 and 7 millions m³, respectively. A flood corresponding to a return period of 100 years coming from the 89 km² basin can be considerably reduced, preventing flood in the downstream areas of the river Wilde Weißeritz. The incoming peak flow of 90.0 m³/s for this event is reduced by 78% to a peak outflow of 20.0 m³/s.

Table 1. Technical data.

Catchment area	89 km²
Reservoir	16 Mio. m³
Height above lowest point of foundation	40 m
Height above valley bottom	33 m
Elevation of dam crest	394 m ü. NN
Crest length	310 m
Crest width	6 m
Raw water delivery	1000 l/s
Minimum water delivery to the downstream river	50 l/s

2 AUGUST 2002 FLOOD EVENT

After nearly a century of operation time, the dam and its installations urgently needed to be overhauled. Besides deteriorations due to the dam's age, mainly on the upstream side of the dam, the outlets and bottom outlets needed to be replaced. Further, the spillway had to be adapted to comply with the new technical standards.

The dam was severely affected by the extreme flood event of August 2002. From the 11th August to 15th August extreme inflows into the reservoir led to very high discharges through the dam's spillway, that seriously damaged it. Additionally, the predam (an earth-fill dam) was overtopped and destroyed.

Due to the high discharge coming from the dam, and additional flows downstream, flood damages of more than 200 Mio € were incurred in settlements downstream of the dam. The extent of the damages would have been more severe without the optimal operation of the hydraulic control structures.

During the 2002 flood event, both the Klingenberg and Lehnmühle dam, and the Malter dam that controls the river Rote Weißeritz delayed the peak flow by 11 hours and as a result the peak flow in the Weißeritz was reduced by 30% (corresponding to almost 140 m³/s). This had a very positive effect for all downstream areas up to Dresden. However, as damages were nonetheless incurred, the normal flood storage room has been increased by 7 millions m³ for all three dams together

The damage caused by the August 2002 flood event (both to the dam itself and assets downstream), made refurbishment of Klingenberg dam urgent and essential.

Figure 1. Water discharge over Klingenberg dam during flood event in August 2002.

The necessary preliminary works started in 2005. Finally, the refurbishment of the Klingenberg dam itself started at the end of 2009, after emptying the reservoir.

Prior to the refurbishment it was necessary to install an alternative system to supply raw water for more than 350 000 inhabitants in Dresden and connected areas.

To this end, a 3 km tunnel was drilled through the underlying hard rock. The tunnel is being used to transport raw water from the newly built upper pre-dam around the empty reservoir to the waterworks. Upon completion of the refurbishment, the tunnel will become part of the spillway system.

Also, an inspection gallery has been constructed by controlled blasting inside the masonry dam, in order to minimize the duration of the refurbishment activities.

3 ALTERNATIVE WATER SUPPLY SYSTEM

The repair works, especially the installation of the new impermeable up water shield have to be carried out with a completely dry reservoir. In order to secure the necessary raw water supply for two waterworks; one in Dresden and one at Klingenberg dam, an alternative water supply system had to be constructed. This work was completed between 2005 and 2008, before the reservoir was emptied. Since November 2009 this alternative supply system is in use, providing raw water to more than 350.000 habitants for a period of three years. In 2008/2009 the alternative raw water supply system was under trial operation for a year.

The main components of this alternative water supply system are a 3 km tunnel, a new pre-dam and a connection to the Rauschenbach/Lichtenberg dam system.

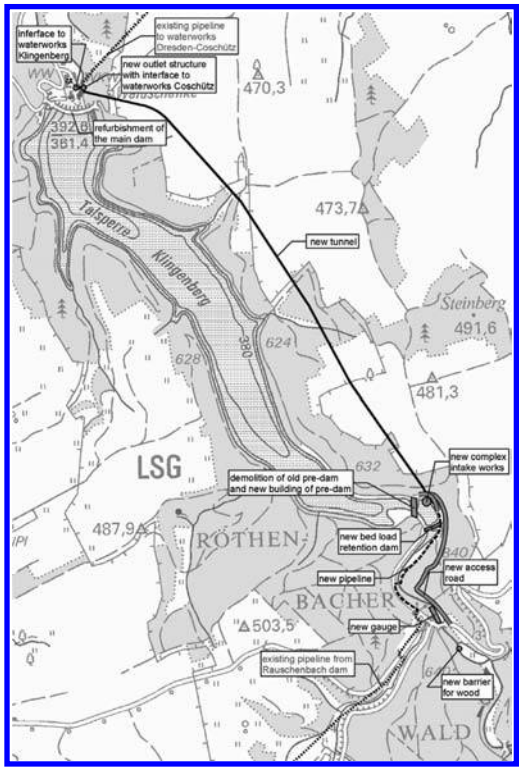


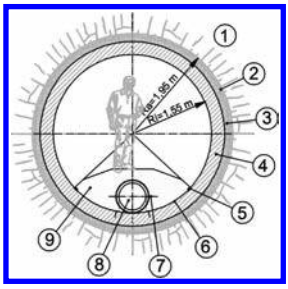
Figure 2. Overview of the complete project construction activities.

3.1 *Tunnel*

The main use of the tunnel during the refurbishment works is for the transport of raw water coming from the upper reservoirs to the waterworks. Since the reservoir was emptied, around 80% of the necessary raw water (up to max. 1, 4 m³/s) is taken out of the new pre-dam and conducted through the tunnel around the empty reservoir and dam, and introduced into the supply system for the Dresden-Coschütz waterwork. A pipework is incorporated in the base of the tunnel, which is directly connected to the Klingenberg waterwork, located immediately downstream of the Klingenberg dam, supplying up to 350 l/s raw water.

After completion of the refurbishment of the main dam, and the new embankment of the reservoir in 2011 the tunnel will be used, together with the predam, to fulfil important water quality and flood control functions, absorbing up to 30 m³/s and bypassing the Klingenberg dam.

It was possible to excavate the tunnel for the Klingenberg dam with a tunnel boring machine for hard rock, because of favourable existing geological conditions.



1 Rock
2 Tolerance tunnel boring machine 5 cm
3 Sprayed concrete d = 10 cm
4 Inner shell, reinforced concrete dmin = 25 cm
5 Sealing profil
6 Isolation film
7 Ductwork for cables
8 Pipeline DN 500
9 Make-up concrete

Figure 3. Cross section of the tunnel.

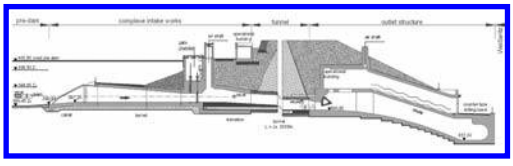


Figure 4. Longitudinal section of the tunnel, with intake and outlet installations.

The tunnel is coated with a concrete shell, and can be used both in free-flowing conditions and under pressure. It has a length of 3350 m, an inner diameter of 3.1 m and a slope of approximately 0.8%.

The raw water is introduced into the tunnel by means of a complex extraction tower located upstream of the predam. The water coming from the Rauschenberg dam, conveyed in a pipeline of 0.5 m diameter is introduced at the extraction tower directly into the tube located at the base of the tunnel, flowing under pressure.

The tunnel ends downstream of the main dam in a counter type stilling basin. The raw water coming from Rauschenberg dam is extracted just above the high head redial gate.

3.2 *New pre-dam*

The old pre-dam, built in the year 1954, had a storage volume of 50.000 m³ and its main purpose was to hold back sediments and increase the water quality in the main reservoir. During the august 2002 floods this pre-dam was seriously damaged, and temporarily repaired in 2004. But to secure the alternative water supply system from 2009 onwards, in 2006 the old pre-dam was demolished and replaced with a new pre-dam. This new pre-dam complies with the new technical standards and meets the necessary hydrological requirements.

The new dam was built as a homogenous earth dam with a height of about 12 m above the foundation and a crest length of 144 m. The material volume employed was 25.000 m³.

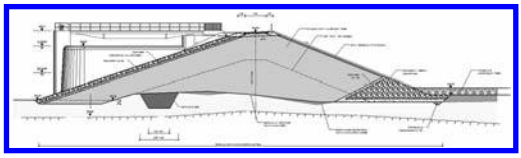


Figure 5. Cross section of the new pre-dam.

The spillway of this new pre-dam is initiated by a 10 m high duck bill spillway. By increasing the height by 4 m compared to the old pre-dam, with the aid of a sliding gate, its operative volume is increased from 50.000 to 230.000 m³ while the alternative water supply system is in operation. Upon completion of the rehabilitation works on the main dam the original retention water level elevation can be restored. The additional storage volume can be used either as flood retention room or to for the temporary storage of water with high contents of sediments, before passing it to the tunnel, in order to maintain the required water quality in the main reservoir. Two bottom outlets with a nominal diameter of 800 mm were included in the front of the spillway.

After elevating the crest line by 4 m the old access road could not be used further. Therefore, a new flood safe access had to be constructed. For the same reason the old gauging station had to be erected at a flood safe position. Additionally, two oil barriers and a floating element barrier were installed, to secure the water quality during the operation of the alternative water supply system and increase the operational reliability of the whole system.

4 REHABILITATION AND REPAIR WORKS

4.1 *Increase of spillway capacity*

During the 2002 flood event the spillway and the stilling basin were seriously damaged. The reservoir was almost completely filled (6 cm freeboard), and the resulting discharge of about 160 m³/s, clearly exceeded the design discharge of 90 m³/s.

This flood event showed that the capacity of the existing spillway system was not sufficient. A new evaluation of the hydrological data, in accordance with required technical standards, clearly showed higher design discharges for the reservoir (design discharge = 145 m³/s and extreme discharge = 225 m³/s).

Taking into account the requirements set out in DIN 19700, the dam had to be adapted to the new hydrological conditions. This adaptation was considered part of the necessary reparation works at the dam.

The spillway was temporary secured, until the complete construction works could be carried out. The goal was to secure the steps of the spillways cascade and restore temporarily the original hydraulic system.

The design plans for the new spillway were based on physical model tests (Thorwarth, J. et al., 2007). In the model all relevant hydraulic parts of the spillway and the dam were reproduced at a scale of 1:30, and evaluated with their current shape and their new design shape. The designed installations could be optimized and the cost-effectiveness of the constructing works improved.

Figure 6. Damages on spillway after August 2002 flood.

Figure 7. Temporarily secured cascades of the spillway.

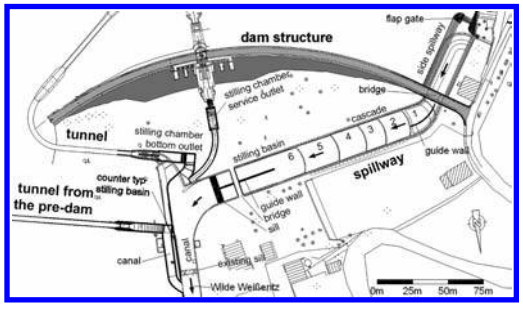


Figure 8. Installations of the spillway system-layout.

Figure 9. Spillway—Physical model and reality.

The constructing works concerning the spillway were initiated in October 2008, immediately after the completion of the tunnel.

The phasing of the rehabilitation works of the dam from the other works made it possible to partially fill the reservoir during this period, and generally simplified the works. Another objective of the separate execution of this works was to fulfil the condition of having a completely empty reservoir for a maximum period of two year.

The main components of the construction works at the spillway were fully designed with the help of physical model research and included:

- Prolongation of the overflow spillway and inclusion of a flap gate, deepening of the collection and drainage channels;
- Rearrangement of the spillways cascade;
- Inclusion of guide walls;
- Elevation of border walls;
- Rearrangement of the stilling basin;
- New bridge over the stilling basin.

All these works had to be carried out taking into consideration that the Klingenberg dam is protected under the Monument Protection Act. This imposes special requirements on the works. For example, the natural gneiss stone had to be installed according to the original joint pattern.

The Works on the spillway were concluded in December 2009.

4.2 *New inspection gallery*

The Klingenberg dam was erected without an inner inspection gallery. During almost 100 year of operation the dam behaviour was monitored and controlled through measurements carried out from

the crest of the dam or the gallery leading to the bottom outlet.

As part of the refurbishment of the dam a new inspection gallery was constructed. Upon completion of the gallery and installation of monitoring equipments, the dam will be upgraded to meet the current technical standards. By means of the instrumentation, it will be possible, for example, to control the impermeability of the new coating (see 4.3) by sectors.

The retrofitting of inspection galleries in high masonry dams is also a requirement of the standard DIN 19700-11.

To simplify the construction sequences, the construction of the inspection gallery was already carried out during the years 2006 and 2007. The new inspection gallery is 210 m long and has 4 access points with a total length of 40 m. The gallery is parallel to the dams' axis with a minimum distance of 5 m from the waterside of the dam. The blow out of the 2 m wide and 2.9 m high cross section was carried out by blasting under normal operational conditions with a completely full reservoir. An independent expert was employed to oversee the blasting works, to ensure that it occurred in a controlled manner.

The quality assurance plan for the blasting was set based on experience gained from other retrofitting construction works of inspection galleries. Most importantly, the maximal oscillation velocity limits of 200 mm/s at 1 m distance from the working face and 100 mm/s at 2 m distance were largely observed.

Prior to beginning the works the contractor delivered a blasting programme with a specified blasting pattern. After undertaking three trial

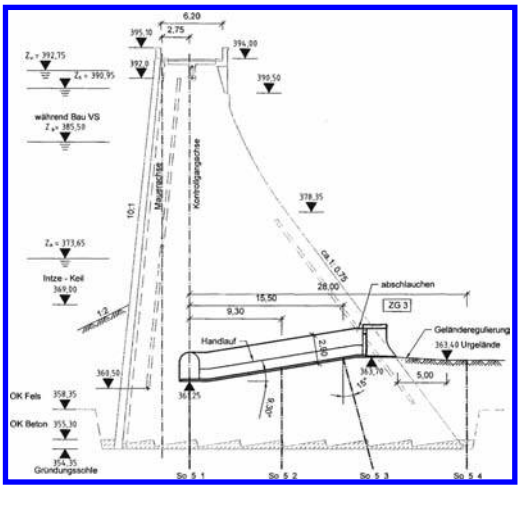


Figure 10. Cross-section of Klingenberg with access tunnel to inspection gallery.

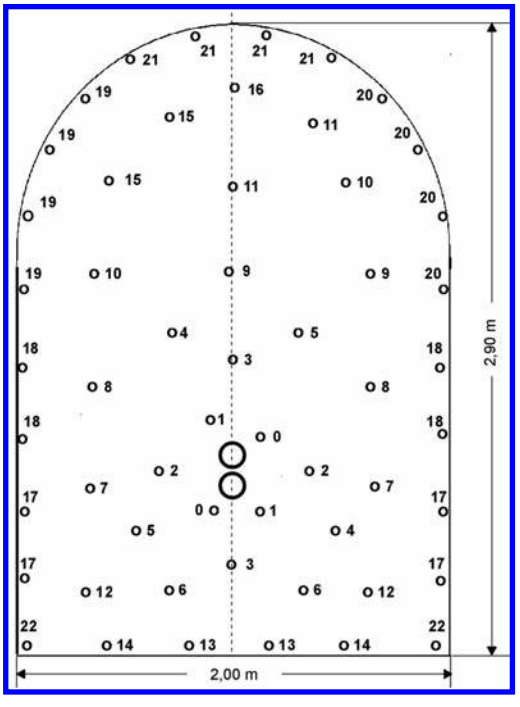


Figure 11. General principle of blast holes.

blasts this pattern was adapted to the local conditions. Following from these preliminary tests, a blasting pattern comprising 55 blast holes, 20 detonation time steps and 2 big-size drill holes was used.

The blasting was undertaken with gelatinous explosive, with a shot length of 1 m. The extraction was facilitated by two big-size drill holes, and was carried out spirally around the blast holes with different time steps. The profiling of the inspection gallery shape was achieved using detonation cord in the border drills, set 30 to 40 cm apart. For each shot length 55 to 72 border drills were drilled.

To control both the near and far area during the blasting, 9 monitoring points were equipped with geophones, two of which were mobile points installed in the nearer environment of the blasting face to measure oscillation as blasting progresses. Further monitoring instruments were installed on the dam crest and in the surroundings of the dam.

Having carefully assessed the conditions of the dam, it was concluded that the blasting technique employed, in accordance with Kittler et al., was adequate for this dam. The limiting oscillation velocity was mostly observed. On occasions in which the limit oscillation velocity was superseded the blasting face and the drill holes were inspected and no damages could be observed. This also provided a good opportunity to study the dynamic

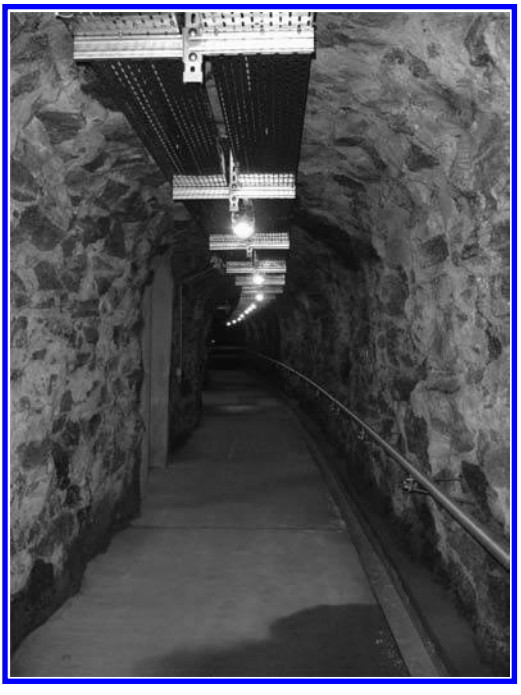


Figure 12. New inspection gallery.

behaviour of the dam and verification of important parameters of the safety calculations.

4.3 *Refurbishment of the main dam*

The refurbishment of the main Klingenberg dam, which is still ongoing, began in October 2009. As it was necessary to completely empty the reservoir during the refurbishment of the upstream face of the dam, a very stringent work plan was established.

The following works are planned for the main dam (some of the activities are currently being implemented):

- Relocation and increase of the capacity of the bottom outlets from 2×8 to 2×15 m³/s by reconstruction and modification of the old bypass;
- Demolition and renewal of the upstream impermeable coating and associated drainage system;
- Demolition and reconstruction of the intake tower, including renewal of all technical installations;
- Conversion of the bottom outlet into a service outlet with an underground stilling basin;
- Partial/complete demolition and reconstruction of the dam crests, taking into consideration the conditions imposed by the Monument Protection Act;

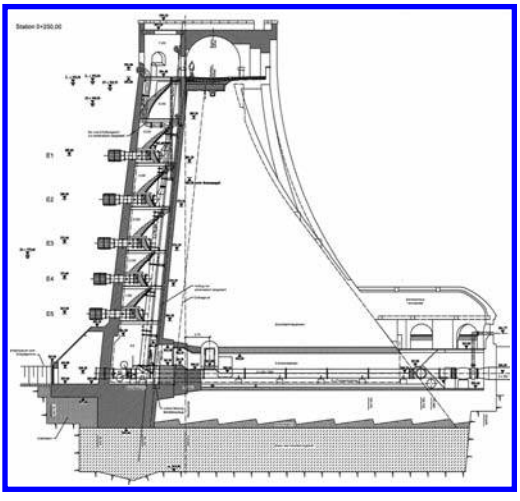


Figure 13. Cross section of Klingenberg dam with intake tower.

- Conservation works on the downstream face of the dam;
- Completion of the inspection gallery;
- Renewal of the monitoring system of the dam;
- Renewal of the electrical and operational installations of the dam.

All works should be finished by the end of 2013.

5 CONCLUSIONS

The Klingenberg dam is currently being refurbished to enhance its structural integrity and upgraded to meet the required technical standard prescribed for dams and reservoirs. In order to implement the refurbishment works, the reservoir had to be emptied and the complex construction work had to be appropriately phased to ensure completion of the construction works within three years, whilst simultaneously reducing the period during which the reservoir remains empty and maintaining continuous supply to its service treatment works.

An alternative water supply system linking the Klingenberg system with its service treatment works was required to ensure continuity of water supply during the construction period. The alternative water supply system was a great challenge to the refurbishment of the Klingenberg dam. The installations that were implemented during this phase will be put to further use after completion of the refurbishment of the dam.

As part of the dam refurbishment the spillway was redesigned and adapted to a new design capacity enabling the dam to withstand floods similar to

the August 2002 flood without sustaining damage whilst reducing the risk of flooding to downstream settlements.

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Physical and numerical modelization of the behaviour of rockfill dams during overtopping scenarios

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ABSTRACT: In recent years the technology on embankments dams has developed sensibly due to the advances in soil mechanic, and in all related sciences. Nevertheless their vulnerability to overtopping is still an open theme of discussion. Such an extreme phenomenon can lead to catastrophic consequences with loss of stability and failure of the entire structure. In spite of that, the consequences of an overtopping are still difficult to analyze and to prevent optimally. The possibility to define a numerical instrument to provide support for analyzing the failure of a dam is an important step ahead for organizing the intervention measures and for optimizing the economic plan.

This is the principal reason why the Spanish ministry of Science and Innovation is funding the XPRES project, between the Polytechnic University of Madrid (UPM), the Centre for Hydrographical Studies of CEDEX and the International Centre for Numerical Methods in Engineering (CIMNE).

The characterization of the failure process is dealt with by combining new numerical models, developed by CIMNE, with experimental tests on physical models of different sizes and scales carried out by UPM and CEDEX.

In the present paper the work carried out by CIMNE is presented. The numerical model is based on combination of Lagrangian Particle Finite Element Method (PFEM) and Eulerian level set techniques in order to optimize the coupling between the dynamic effect of water inside and outside the dam, with the structural deformation and collapse induced on the down stream shoulder. The breaching formation and the consequent beginning of the failure are analyzed with particular interest. The specific features of the adopted method make it appropriate to treat the rockfill material and its large deformations and shape changes. A variable viscosity law and an energetic erosion criterion are used to take into account mass sliding and erosion acting in combined or alternative way.

1 INTRODUCTION

The rehabilitation of existing dams and their safety analysis are nowadays open fields of research. An increasing interest is rising on the study of rockfill embankment dams during overspilling. In fact in many countries of the European community for instance, design criteria of this kind of structures have recently been reviewed with the intention of increasing safety level in front of an exceptional flooding. That means that many existing structures have now to be modified. The planning of these interventions is actually limited by the lack of knowledge on the failure process or, more in general, on extreme conditions.

The possibility of studying the failure process is currently limited by the lack of a precise knowledge and by the absence of a suitable computational method which can predict accurately the onset and evolution of breaching. The analysis of the possible consequences of an accidental overspill is then impossible or very imprecise and the necessary economical measures for solving the problem are then inefficient. An appropriate computational method will help to reduce the economic impact of the investments in dam safety and in emergency plans for embankment dams.

The dam failure is mainly due to the hydrodynamic forces acting on the rockfill material in the downstream face. Two failure mechanisms occur in an alternative or combined mode: dragging of rockfill particles with erosive effects, and loss of stability of a part of the downstream region due to the slippage of a bulk mass of the rockfill material.

The loss of a relevant part of the rockfill material in the downstream side of the dam, can lead to mechanical failure of the impervious region, either if this is formed by a concrete (or asphalt) face, or by a clay core. In the latter case, erosion

of the cohesive material can also influence the failure process, but the scale time of appearance of this erosion phenomenon is so different from the principal erosion of the rockfill part, that it is not taken into account. The objective of the current work is to develop and validate a new computational method of general applicability that allows treating the above problems. The method will combine advanced finite element and particle techniques. Fluid-structure interaction effects and non-linear geometrical and mechanical effects in the dam material will be considered. The specific features of the proposed method make it appropriate to treat the rockfill material adjacent to the downstream surface where a transfer of momentum between the water flowing over the dam surface and that moving through the interior due to seepage occurs.

2 EMBANKMENT DAMS

In recent years technology on embankments dams has developed a lot due to the advances in soil mechanics knowledge, and in all related sciences. This aspect, combined with the evident economic advantage of construction, make often this kind of structure a more appealing choice with respect to traditional concrete dams. The design of earth dams is very flexible and makes use of many different materials, that can often be found *in situ*, and shapes. They can have for example an impermeable core or an external impermeable covering.

Due to the recent advances in geothecnical field, the tallest dam in the world are nowadays earthdams (i.e. Rogún dam (335 m) or Nurek dam (300 m)) and their number overcomes the classical concrete structure, as can be seen in Table 1.

Nevertheless their vulnerability to overtopping still remains their weakest point in comparison with concrete structures. When the water exceeds the crest of the dam, the consequences can be catastrophic. An exceptional flooding compromises seriously the structure, leading, in almost all cases, to its failure. Among others, we recall the failure of the Tous dam in Valencia. In October 1982, a tsunami of 20 million of m^3 of water flowed through the *Comunidad Valenciana*.

Table 1. Distribution of the different types of dams. Data taken from ICOLD bulletin (99).

DAM	N of dams	%
Earth dams	28.131	77,6%
Rockfill dams	1.843	5,1%
Gravity dams	4.180	11,5%
Others	2.081	5,4%

Following the ICOLD bulletin, while in concrete dams the cause of the failure of the structure is due to problems with the cement in 42% of cases, in the case of dams made of incoherent material, overtopping is the most frequent cause of failure (31% as principal cause, 18% as secondary cause).

Embankment dams are very vulnerable to overtopping phenomena because of their particular nature, being made of incoherent material. The failure mechanisms in fact, are mainly due to the hydrodynamic forces acting on the rockfill part in the downstream slope. These forces cause a huge loss of material which may trigger the instability of the entire dam. They can be resumed by two kind of phenomena, acting in combined or alternative way: dragging of rockfill particles with erosive effects and loss of stability of a part of the downstream region due to the land slide. The loss of a relevant part of the material in the downstream part can carry to the failure of the entire dams.

3 THE EXPERIMENTAL ANALYSIS CARRIED OUT BY UPM AND CEDEX

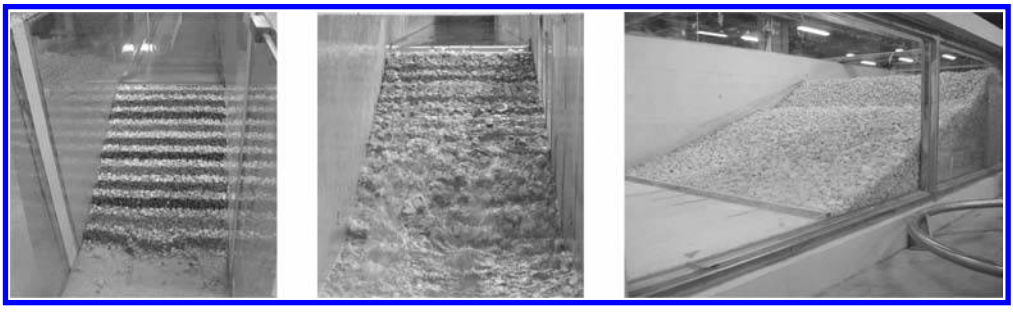
UPM and CEDEX have carried out more than 70 experiments during the last two years. They analyzed the influence of a series of parameters on the failure mechanisms. These parameters are, for instance, the dimension of the rocks, the slope of the downstream part of the dam, the type of impermeabilization used, and so on.

All the experiments were performed in three channels of different dimentions in order to be able to study also the scale lenght effect. The three different experimental settings used are presented and described in Figure 1. For more details on the experimental campaign see the paper *Initiation phase of rockfill dams failure due to overtopping*, presented to this conference by M.A. Toledo and coworkers.

4 THE NUMERICAL MODEL

The principal aim of this work is the creation of a tool able to simulate the initial stage of failure of the dam when an overtopping or an exceptional flood occurs. Two different models are developed to study the water behavior and the seepage line variation and, on the other hand the structural response to such dynamic action. Fluid and dam model are finally coupled in order to consider the interaction and reciprocal influence of the two media.

Both the water and the dam are studied using a continuum mechanics approach. The variables of the problem are then the Darcy velocity, averaged over the total volume, water pressure and velocity



(a) Small channel. 0.4 m width, 0.6 m height, 12 m long

(b) Medium channel. 1.0 m width, 1.1 m height, 16 m long

(c) Big channel. 2.48 m width, 1.4 m height, 13.7 m long

Figure 1. UPM and CEDEX experimental channels.

and pressure of the dam, considered as a continuum. This implies that a single element does not coincide with a granular particle but with a cluster of particles.

In the following sections a schematic overview of the numerical approach is presented separately for the fluid and the structural model. First the fluid model is analyzed and some validation results are presented. The structural model is then described focusing in particular on the choice of the dam constitutive model and the numerical treatment of the two main failure mechanisms: erosion and mass sliding. Finally some preliminary results of the coupled model are shown.

4.1 *Evolution of seepage*

Classical studies of fluid flow into a porous media can't be used for the analysis of the water motion within the dam material. Traditionally water is considered in slow motion or as a stationary load (Zienkiewicz et al. 1999). On the contrary in the case of an overtopping, the possibility to follow the rapid transition of the water level in the downstream slope is a key point for the identification of the beginning of the failure mechanism. On the other hand the typical problem of evaluating the saturation level loses its importance in the case studied, due to the dimension of the granular material. At a *micro* level the flux between the rocks can be assimilated to an open channel flow with localized turbulence phenomena. This behavior is reflected, at a *macro* level, by the impossibility of considering the Darcy law in its linear form (Lancellotta 2005). Many quadratic and exponential relations between velocity and pressure gradient are presented in literature: Brinkmann's and Forchheimer's modification of Darcy's law, are two of the most famous theories that consider the non linear relation between the water speed and the hydraulic gradient (Nield and Bejan 1992). Nevertheless two important drawbacks limited their generality: from one hand a variable porosity cannot be considered and, from the other, the flow does not reduce to the free fluid flow when porosity is equal to 1. Following the work of Nithiarasu (Nithiarasu et al. 1997) and (Nithiarasu and Liu 2006), the classical form of the governing equations for an incompressible fluid are here modified, inserting the effect of porosity and the non linear relation given by the Ergun's modification to Darcy's law therefore, ensuring that all the previously presented drawbacks are overcame. The obtained governing equations reduce to the classical form of the Navier-Stokes equations if the porous medium is no longer present allowing the simultaneous treatment of the water behavior both outside and inside the dam (Zienkiewicz et al. 2005).

$$\rho_f \partial_t \mathbf{u}_f + \rho_f \bar{\mathbf{u}}_f \cdot \nabla \mathbf{u}_f + n\nabla p - \mu \Delta \mathbf{u}_f - \rho_f \mathbf{b} n - \mathbf{D} = 0; \qquad \nabla \cdot \mathbf{u}_\mathbf{f} = 0; \tag{1}$$

In eq. 1 $\partial_t \mathbf{u}_f = \partial \mathbf{u}_f / \partial t$., ρ_f, μ are the fluid density and kinematic viscosity, n is the porosity of the medium. p is the fluid pressure and $\mathbf{u}_\mathbf{f}$ is the fluid Darcy velocity. The latter is, by definition, the fluid velocity averaged over the total volume, whereas the fluid velocity $\bar{\mathbf{u}}_\mathbf{f}$ is averaged over the fluid volume. Their relation is stated by the Dupuit-Forchheimer equation $\mathbf{u}_f = n\bar{\mathbf{u}}_\mathbf{f}$ (Nield and Bejan 1992). Finally $\mathbf{D}$ is the Darcy term: it represents the dissipative effects due to the interaction between the solid and the fluid part.

$$\mathbf{D} = \frac{\mu}{k}\mathbf{u'}_\mathbf{f} + \frac{1.75}{\sqrt{150}}\frac{\rho_f}{\sqrt{k}}\frac{|\mathbf{u'}_\mathbf{f}|}{n^{3/2}}\mathbf{u'}_\mathbf{f}; \tag{2}$$

where $\mathbf{u'}_\mathbf{f}$ represents the relative velocity of the fluid with respect to the solid matrix, k is the permeability.

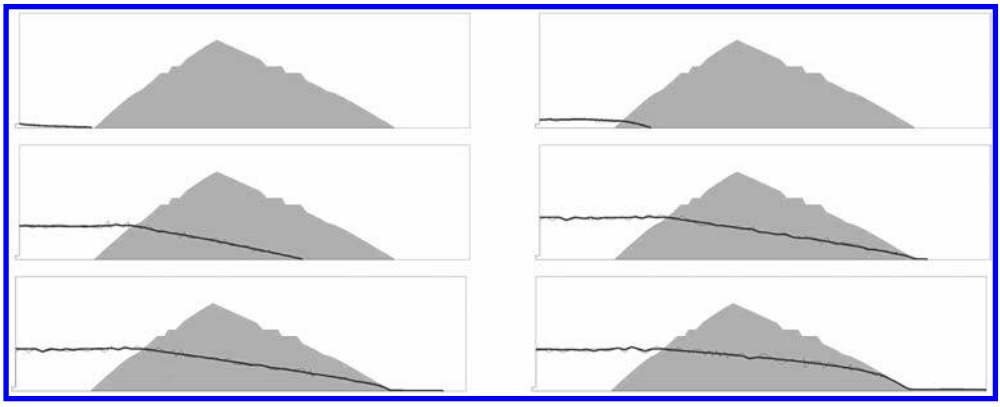


Figure 2. Evolution of the seepage line in a dam with porosity = 0.45 and D_{50} = 1.0 cm. Q = 0.46 l/s.

An Eulerian approach is chosen and a level set method is employed to track the free surface of the fluid. In the Eulerian kinematical description of motion a fixed spatial mesh is considered and the evolution of velocity and pressure at the mesh point is evaluated. In a Lagrangian description of motion, on the contrary, the material particle is followed during its movement (Donea and Huerta 2003). The level set method is a well established technique to follow the evolution of an interface under a velocity field (Osher and R.P. Fedkiw). The free surface is identified as the zero values of a smooth function that takes negative value inside the fluid and positive value outside. The convection of this function, according with the fluid velocity field, allow to follow the evolution of the free surface.

Figure 2 shows the evolution of the seepage line inside the dam and the free surface outside it. The dam is an homogeneous medium with constant porosity 0.45 and D_{50} = 0.01 m the dam is present but not calculated at the moment. It defines nevertheless the region of space with porosity different than 1. An incoming discharge of 0.46 l/s is entering the domain from the left. Pressure distributions a the bottom of the dam have been compared with experimental results considering different incoming discharges as shown in Figure 3.

4.2 *Structural response: The Particle Finite Element Method (PFEM)*

Rockfill collapse is different from that of clay or sandy soils. In the latter, capillary suction is weakened by the presence of water; in a granular material on the contrary, two are the main mechanisms of irrecoverable strains: the sliding between particles and the crushing of particles in contact area in function of the normal forces acting on them. In sand these phenomena occurs only under high stresses, whereas on a rockfill material, due to the big dimension of the particles, even low stresses can be the causes of this strains (for more details see (Anthiniac et al. 1998) and (M.A. Toledo 1998)). The PFEM is a suitable instrument for the calculation of the stress and strain fields inside the dam due to the changing of the pressure values. The specific features of the PFEM make it appropriate to treat the rockfill material adjacent to the downstream surface where a transfer of momentum between the water running over the dam surface and that moving through the interior due to seepage occurs.

From one side the superficial particles are dragged away due to the frictional work of water coming out of the toe, after the seepage process. On the other hand, if the slope is more accentuated, a mass slip phenomenon can be predominant compared with erosive effects. Both phenomena act in combined or alternative way, mainly in function of the geometrical conditions of the slope.

For the treatment of the structural response the particle finite element method is used.

The PFEM is a numerical method that uses a Finite Element mesh to discretize the physical domain and to integrate the differential governing equations (see (Idelsohn et al. 2004) and (Larese et al. 2008)). In the PFEM the domain is modeled using an Updated Lagrangian Formulation. That is all the variables are assumed to be known in the current configuration at time t and they are brought in the next (or updated) configuration at time $t + \Delta t$. The Finite Element Method (FEM) is used to solve the continuum equations in a mesh built up from the underlying nodes (the particles). Hence a mesh discretizing the domain is generated, in order to solve the governing equations in the standard FEM fashion. The nodes can be viewed as material

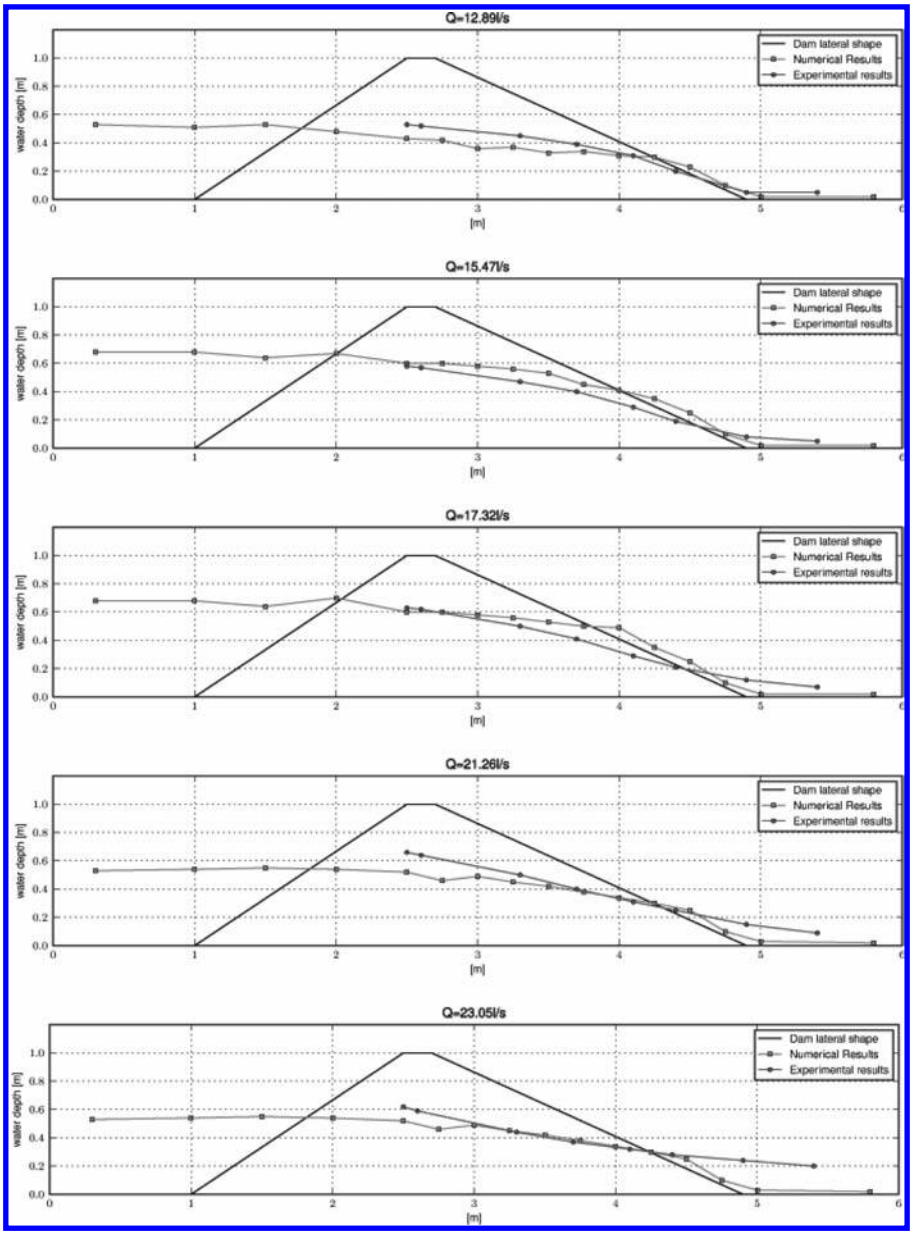


Figure 3. Seepage line at stationary regime. Porosity = 0.425; D_{50} = 1.0 cm.

particles whose motion is tracked during the transient solution. This is useful to model the separation of solid particles from the bed surface and to follow their subsequent motion as individual particles with a known density, an initial acceleration and a velocity subject to gravity forces (Oñate et al. 2004) and (Aubry et al. 2005). It is important to underline that each particle is treated as a material point characterized by the density of the solid domain to which it belongs to. The global mass is obtained by integrating density at the different material points over the domain. The quality of the numerical solution depends on the discretization chosen as in the standard FEM. Adaptive mesh refinement techniques can be used to improve the solution in zones where large gradients of the fluid or the structure variables occur.

4.2.1 *Dam boundary conditions*

The action of water on the dam is a key point in the calculation of the structural response. The projection of the fluid velocity and pressure field together with the Darcy term (see eq. 2) has to be performed as preliminary step in order to define the boundary conditions of the coupled problem. The tool provides a projection algorithm between non matching meshes that can easily transfer information from the eulerian fixed mesh to the lagrangian one.

4.2.2 *The dam constitutive model: Mass sliding*

The rockfill part of the dam is treated as a continuum, as already mentioned in section 4. Many plastic or rigid-plastic constitutive models are commonly used in geomechanics to describe its rigid behavior up to yield stress level and the subsequent failure mode. In the case studied no deformation is present until effective stress is greater than zero. Once yield is achieved a flux of granular material, dragged by the action of water is generated.

This idea of the granular flow leads to concentrate on variable viscosity models of fluids for the calculation of the structural part as well.

Fluids for which the constitutive law cannot be written as $\sigma = -p\mathbf{I} + \tau$ are called Non-Newtonian fluid. In this case viscosity cannot be considered a constant property of the material but it is strictly dependent on the deformation process.

As already stated above, a slope of granular non cohesive material has the capability to support a certain amount of shear stress before starting large deformations. Bingham plastic are a suitable model to describe the dam behavior. They are non newtonian fluids behaving as a rigid body until reaching a yield stress value (τ_0) and following then a viscous behavior like:

$$\begin{aligned} \dot{\gamma} &= 0 \qquad if \quad \tau < \tau_0 \\ \dot{\gamma} &= \frac{1}{\mu}(\tau - \tau_0) \qquad if \quad \tau \geq \tau_0 \end{aligned} \tag{3}$$

where $\dot{\gamma}$ is the rate of strain, μ is the kinematic viscosity and τ the shear stress.

Since the bilinear behavior stated by eq. 3 can induce numerical difficulties, some smoothed laws are usually preferred (Papanastasiou 1987). Nevertheless some authors (Lipscomb and Denn 1984) tried to simulate what is called bi-viscosity model but their predictions lead to inconsistencies. This is the reason why we have adopted the model developed by Papanastasiou (Papanastasiou 1987) based on an exponential evolution of the viscosity.

$$\tau = \left[\mu + \frac{\tau_0}{|\dot{\gamma}|}\left(1 - e^{-m|\dot{\gamma}|}\right) \right] \dot{\gamma} = \tilde{\mu}\dot{\gamma}; \tag{4}$$

Different level of approximation to the bilinear model can be obtained varying the m parameter.

The yield stress τ_0 is not constant in the present study. It depends on the characteristic of the material, on its friction angle. Moreover it has a strong dependency on the relation between the structural pressure and the water pressure. Considering two points with the same value of water pressure, whether the solid pressure is lower (superficial nodes) it will be more easy to separate the solid grains. On the other hand, if the solid pressure is the same, we need a higher value of water pressure to separate the grains. The yield stress is therefore proportional to the difference between the solid and the water pressure.

The resolution system of the dam treated as a Non Newtonian Bingham fluid, is a traditional Navier-Stokes problem for Non-Newtonian fluids

$$\begin{aligned} \rho_s \partial_t \mathbf{u}_s + \nabla p_s - \nabla \cdot (\tilde{\mu} \nabla^s \mathbf{u}_s) - \rho_s \mathbf{b} + \mathbf{D} = 0; \\ \nabla \cdot \mathbf{u}_s = 0; \end{aligned} \tag{5}$$

where $\mathbf{u}_s$ and p_s are the solid velocity and pressure and ρ_s is the solid density. The only peculiarity is that viscosity $\tilde{\mu}$ states for the apparent viscosity presented in eq. 4. As already explained it is not constant but it is influenced by the deformation of the continuum and its non linearity has to be taken into account.

The convective term is not present due to the choice of a Lagrangian description of motion.

The **D** Darcy term, calculated in the fluid formulation (eq. 2), acts as an external force on the granular material and allows the global equilibrium of the coupled monolithic system.

4.2.3 *Superficial dragging*

The possibility of modeling superficial erosion of the particles is very important in the prediction of the evolution of surface material dragged away by hydrodynamic forces. For the simulation of erosion we propose a model based on the frictional work at the bed surface originated by the shear stress in the fluid using Particle Finite Element Method (PFEM). This technique, has already been successfully used in (Oñate et al. 2008) and (Oñate et al. 2006).

In the continuum approach each node is not representing a single grain but a cluster of particles. Using solid transport theories, a value of critical fluid velocity can be calculated for a given material. When a superficial node is invested by a flux of water whose velocity overcomes this limit, the node starts accumulating frictional work. When the amount of energy overcomes the critical frictional work necessary to move all the volume directly "attached" to the node, then it changes its

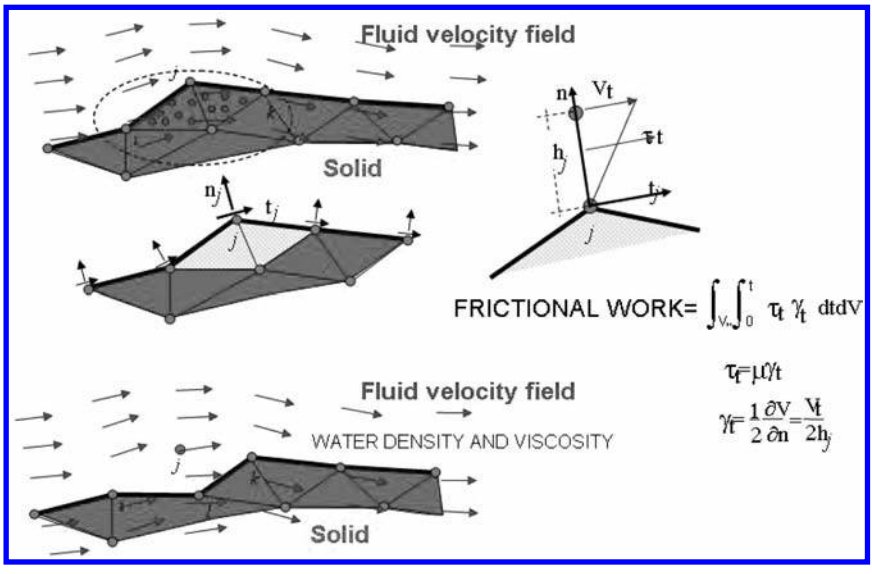


Figure 4. Schematic representation of numerical treatment of erosion.

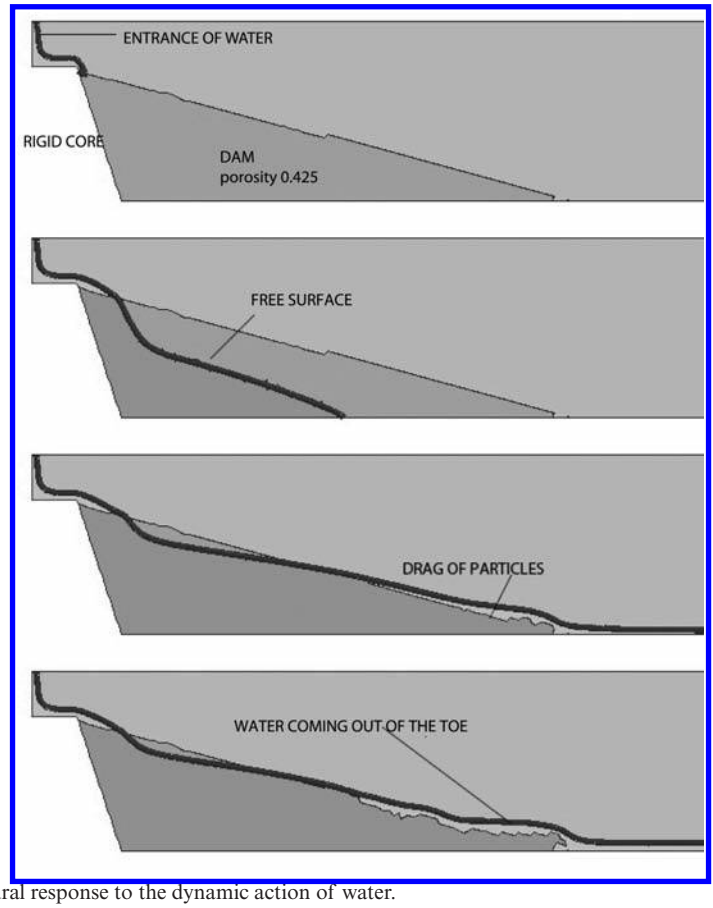


Figure 5. Structural response to the dynamic action of water.

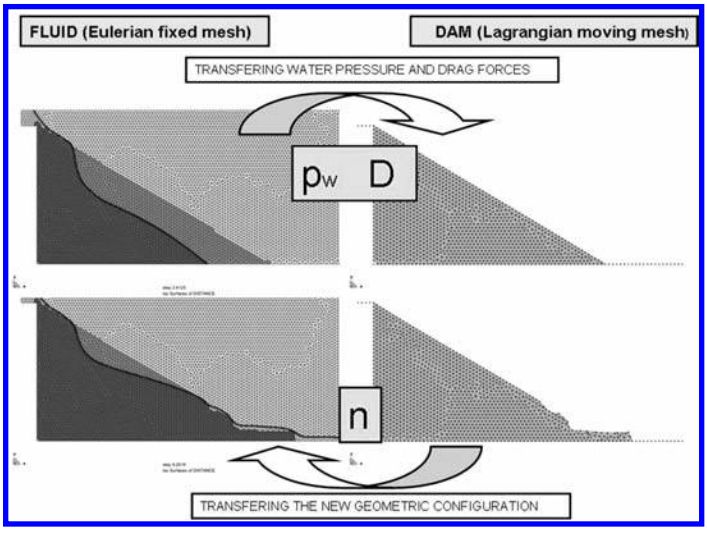


Figure 6. Summary of the basic steps of the global numerical algorithm.

material characteristics and starts flowing under the action of water.

5 COMBINATION OF THE FLUID AND DAM MODEL

The dam failure is mainly due to the hydrodynamic forces acting on the rockfill material in the downstream shoulder. As already said these actions have a double effect: the dragging of rockfill particles with the corresponding erosion, and the loss of stability of a part of the downstream region due to the slippage of a bulk mass of the rockfill material. The two failure mechanisms occur in an alternative or combined mode. The loss of a significant mass of rockfill material in the downstream face can lead to mechanical failure of the impervious region, either if this is formed by a concrete or an asphalt face, or by a clay core. An accurate coupling of all the different aspects of overtopping is needed to simulate the whole phenomenon of failure. One of the possibilities in Fluid Structure Interaction (FSI) analysis is to construct a unique monolithic system where all the equations, those of the fluid, those of the solid and those of the coupling part, are solved at the same time. The excessive cost of this approach make often this choice inadequate. Usually staggered schemes are preferable although less accurate.

In this particular case, the use two different solution methods for the dam and for the fluid leads implicitly to a partitioned scheme. The coupling is done through the Darcy term (2) which is evaluated in the fluid field and projected on the dam where it acts as an additional external force. On the other hand the dam configuration (i.e. the porosity distribution in space), is evaluated in the PFEM model and projected then to the eulerian mesh in order to define the new boundary conditions for the fluid problem.

The flow chart of the entire simulation process can be schematically summarized by the following steps:

1. **SOLVE** the *water* free flow and seepage problem calculating VELOCITY and PRESSURE field in an EULERIAN fixed mesh;
2. **Project** the FLUID VELOCITY and PRESSURE field on the dam;
3. **Project** the non linear DARCY TERM on the dam (it will act as an additional external force);
4. **CALCULATE** the *structural response* in terms of STRESS and STRAIN in a lagrangian mesh, using PFEM;
5. **Project** the new configuration of the dam in terms of POROSITY distribution on the Eulerian fluid domain;
6. **Project** the DAM VELOCITY on the Eulerian fluid domain (to calculate the Darcy term);
7. Go back to step 1.

Figure 6 presents a scheme of the algorithm.

6 CONCLUSIONS

The analysis of extreme phenomena on rockfill dams is still an open issue due to the difficulty of forecast of the failure process. This problem is complex both from theoretical and computational

point of view and this explains why its solution, using reliable and accurate methods, has not been possible up to now.

In the present paper an innovative numerical approach to simulate the consequences of an overtopping on embankment dams and the initial failure mechanism is presented.

The fluid behavior is analyzed on an Eulerian fixed mesh. The traditional Navier-Stokes Equations are modified in order to take into account the presence of porosity and de Darcy law, in its non linear formulation.

The dam response is evaluated on a Lagrangian moving mesh using PFEM. A Bingham Non-Newtonian law is used to describe the structural behavior and collapse of rockfill under the effect of water. An energetic criteria is used to evaluate the superficial water erosive effect.

A projection algorithm allows the transferring of information through non matching meshes.

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Renovación de equipos electromecánicos en desagües profundos

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ABSTRACT: There are a big number of dams in Spain who are older than 40 years. The layout of the low level intakes is very different. The types of gates are also varied. During the last 30 years Ingeniería y Construcciones de Sur, S.A. (INGESUR) has been working at dams with low level intakes using a wide range of solutions for repairing or erection of new valves and gates. The present paper pretends to show a non exhaustive view of these solutions but to give an idea of the possibilities available to update these mechanical elements in dams.

Como es sabido, el parque de presas existente en España está constituido por un gran número de ellas que supera con creces los 40 años. La tipología de los Desagües Profundos de estas presas es muy variada en cuanto a la configuración de los órganos de guarda y regulación. En ellos nos podemos encontrar válvulas mariposa, válvulas de compuerta, compuertas tipo Bureau, válvulas Larner-Johnson, válvulas Howell-Bunger, compuertas Taintor de fondo, etc.

La disposición del conducto completo también puede presentar configuraciones variadas: existencia de rejas a su vez con distintas configuraciones, embocaduras planas, curvas, dobles, asimétricas, conductos rectangulares, circulares, túneles de acceso hasta la compuerta de aguas arriba de cientos de metros, túneles de salida de agua de cientos de metros.

En determinados casos los caminos de acceso, las galerías y las cámaras de compuertas de la presa pueden constituir o representar una dificultad para la renovación de estos órganos de desagüe, impidiendo su desmontaje o el acceso de los nuevos equipos.

En los 30 años que Ingeniería y Construcciones del Sur, S.A. (INGESUR) lleva trabajando en estos campos, hemos renovado multitud de órganos de desagüe profundo con soluciones adaptadas a cada caso particular.

Soluciones de todo tipo como colocación de escudos mediante equipos de submarinistas, utilización de la compuerta de aguas arriba como escudo, colocación de nuevos órganos de regulación aguas abajo de los existentes, desmontaje y reparación de compuertas y válvulas, sustitución de accionamientos oleohidráulicos o mecánicos, reparación de cierres en cuerpos de válvulas hormigonados, modificación de aducciones de aire, sistemas de by-pass son ejemplos de una amplísima casuística sobre la que haremos un repaso mediante esquemas, planos y fotografías.

1 INTRODUCCIÓN

De acuerdo con la Guía Técnica de Seguridad de Presas nº 5 "Aliviaderos y Desagües", los desagües considerados profundos son aquellos "situados en los dos tercios inferiores del embalse", haciéndose la distinción entre tomas de explotación, desagües intermedios y desagües de fondo.

El título de la presente ponencia podría haber sido "Renovación de Equipos Electromecánicos en Desagües Sumergidos", dado que la renovación de estos equipos en tomas que quedan durante meses en seco puede ser considerablemente más sencilla de lo que aquí se expondrá. Las diferentes soluciones que se exponen son para desagües inundados donde se necesita disponer de seguridad aguas arriba para poder trabajar sin que el embalse se vacíe hasta la cota del desagüe.

2 PROBLEMÁTICA GENERAL. ENVEJECIMIENTO DE LAS PRESAS Y FALTA DE USO

Si pensamos que una presa construida en 1980 tiene ya 30 años de antigüedad y nos hacemos una idea de cómo de antigua es cualquier máquina en una fábrica, un automóvil o un electrodoméstico, con 30 años, podemos imaginar muy bien el estado en que pueden encontrarse muchos equipos electromecánicos en las presas cuando muchas de ellas superan los 50 años de antigüedad. Muchos de estos equipos, además, no han tenido ningún tipo de mantenimiento o reparación desde que se instalaron o, simplemente, como ocurre también, no se

han maniobrado en años. Los motivos de esta situación pueden ser diversos, pero la escasez de presupuesto, la falta de personal o, sencillamente, el desconocimiento, suelen ser los más frecuentes.

Así, encontramos en las presas compuertas de seguridad que no son capaces de abrir o cerrar, o que sólo lo hacen con presiones equilibradas. Accionamientos oleohidráulicos que a cada maniobra pierden decenas de litros de aceite, enclavamientos que no funcionan, compuertas que se bajan o se abren solas. A veces, equipos que, ya nadie sabe maniobrar y de complejo mantenimiento, como ocurre frecuentemente con las válvulas Larner-Jonhson. Nos encontramos con conductos aterrados, fugas muy grandes en las compuertas, conductos de by-pass que no se pueden cerrar, tubos de desentarquinado conectados directamente al vaso del embalse que se pudren, aducciones de aire insuficientes, vibraciones, etc.

Normalmente el problema más acuciante ocurre cuando el equipo responsable de la presa, la empresa de mantenimiento o, incluso, fabricantes, como nosotros, temen, incluso, realizar la maniobra de apertura de la compuerta ante la posibilidad de que no se pueda cerrar después, con el riesgo de vaciado del embalse hasta esa cota. Esta situación provoca que la compuerta no se maniobre nunca acentuando aún más sus problemas de funcionamiento y lo que puede ser aún peor, provocando el aterramiento de la toma en cuestión.

Otros casos que se pueden dar es que, a pesar de estar los equipos perfectamente conservados o mantenidos se requiera una mejora en el sentido de utilizar válvulas o compuertas más adecuadas para la regulación o simplemente más fáciles de mantener.

3 CONFIGURACIONES DE SOLUCIONES

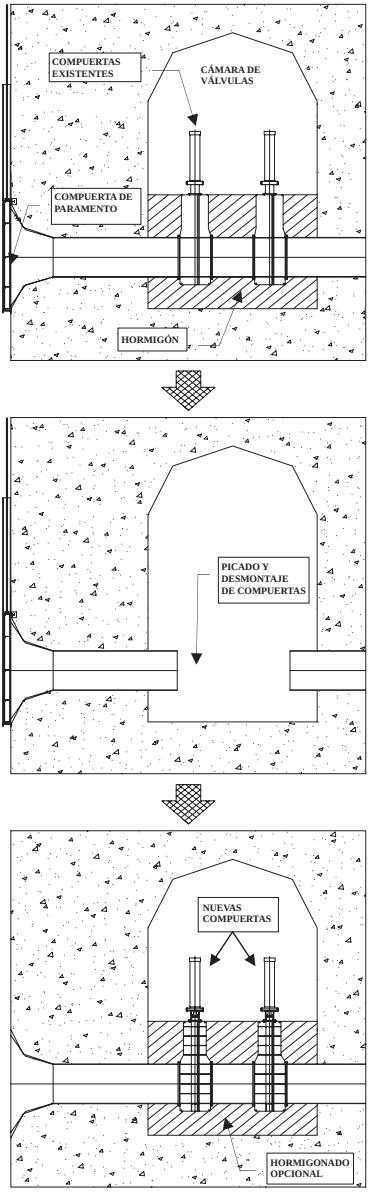
A continuación expondremos un abanico de soluciones que, sin pretender ser exhaustivo, puede dar una visión de las posibilidades que existen.

3.1 *Compuerta de paramento como elemento de obturación del conducto*

Una configuración muy común del desagüe, sobre todo en presas antiguas, es el disponer de una compuerta de paramento en el inicio del conducto por aguas arriba dentro del vaso del embalse.

Esta compuerta puede ser de grandísima utilidad dado que si se puede cerrar, no necesariamente cortando el pleno caudal, sino sólo el caudal de fugas, presentando a su vez fugas que no sean demasiado considerables, es posible retacar el tablero mediante equipos de buceadores profesionales, permitiendo los trabajos en las compuertas de aguas abajo. En el caso de que las fugas fueran tales que no se pudieran acercar los buceadores por el temor de ser succionados, una estrategia posible es la de tratar de minimizar las fugas en la compuerta más aguas abajo, por ejemplo, con la colocación de planchones de goma en sus zonas de cierre u otras en función de la configuración que exista.

El gráfico reproduce una disposición clásica de una línea de un desagüe con dos compuertas tipo Bureau (o podría ser con dos válvulas de compuerta o con válvula de compuerta y válvula mariposa).



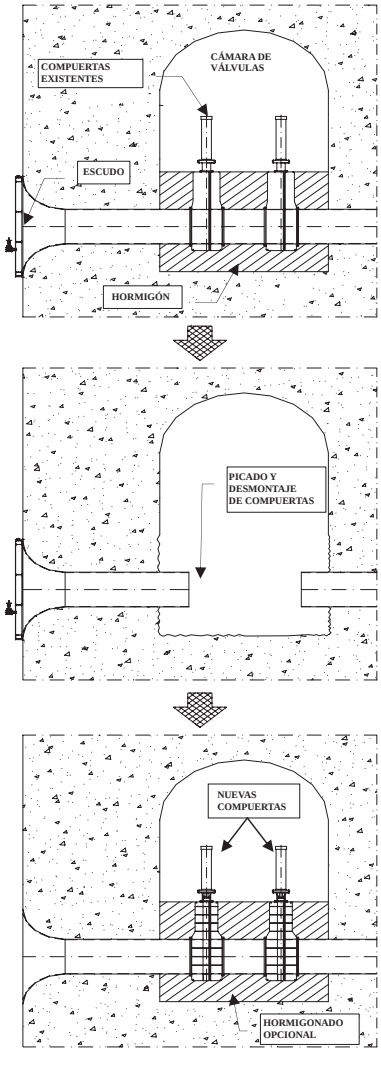
Una vez que la compuerta de paramento permite trabajar aguas abajo, se procede a la demolición del hormigón, si existe, y a la retirada de las viejas compuertas y su sustitución por otras nuevas, en este caso hemos representado esquemáticamente dos compuertas Bureau.

Si las dimensiones lo permiten, se pueden reponer los conductos hasta la embocadura y hacia aguas abajo de las compuertas.

3.2 *Escudo como elemento de obturación*

Este caso se diferencia del anterior en que no se dispone de compuerta de paramento.

En este caso se necesita la colocación de un escudo que permita manipular en la compuerta de aguas arriba. Por escudo entendemos un elemento de obturación, normalmente metálico, dotado de bandas de goma para asegurar el cierre. Este elemento se fija al paramento de la presa simplemente por diferencia de presiones entre aguas arriba y aguas abajo. Para su retirada se procede al equilibrado de presiones, quedando el escudo libre.



Para poder realizar la colocación de un escudo se requiere una superficie de apoyo homogénea y capaz de resistir las, normalmente, elevadas presiones que se producen en las zonas de cierre.

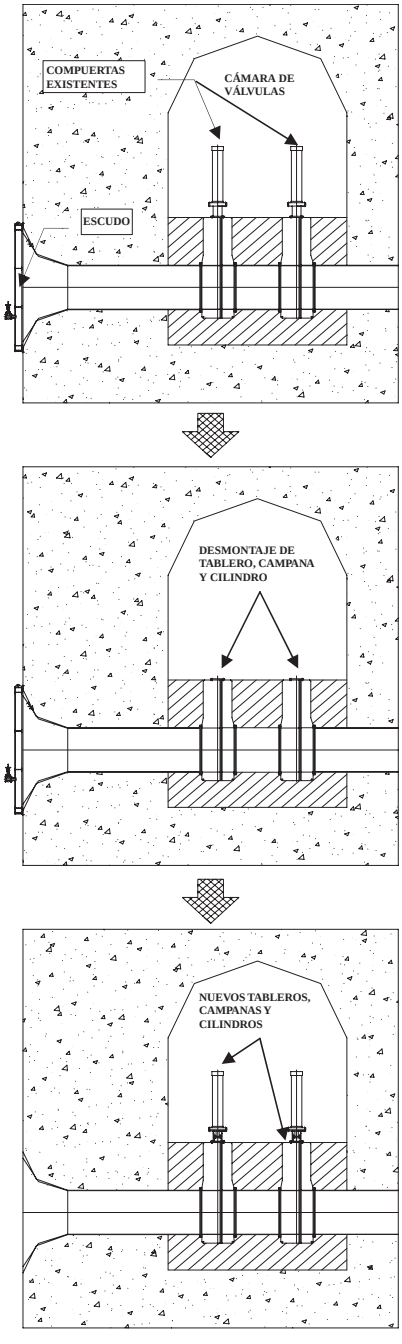
La tipología de escudos más típica es un escudo plano que puede ser flotante o no en función de que la embocadura a obturar esté al alcance de una grúa o no. En embocaduras troncocónicas pueden usarse escudos también troncocónicos, si bien se ha de ser cuidadoso dado que estos escudos pueden "clavarse" y no ser suficiente el equilibrarlos en presiones para separarlos de su punto de obturación.

3.3 *Escudo y cambio de campana, tablero y accionamiento*

Cuando se dispone de dos compuertas tipo Bureau o dos válvulas de compuerta hormigonadas en una cámara, puede ocurrir que el esfuerzo económico y de ingeniería necesario para picar y extraer el material, así como la dificultad de introducir a través de angostas galerías una nueva compuerta, sean tan elevados, que se desista de reparar el desagüe.

En muchas situaciones, sin embargo, hemos conseguido simplificar este proceso, conservando los cuerpos hormigonados de las compuertas, normalmente Bureau, que, en general, están muy bien conservados.

Lo más habitual es que, en el cuerpo fijo, tanto las bandas de cierre laterales como de dintel puedan estar deterioradas y el cierre inferior destruido. Cuando esto ocurre, se extrae el tablero, la campana y el cilindro (o mecanismo de husillo eléctrico), que son los elementos que pueden estar más deteriorados o estar obsoletos, se toman



medidas cuidadosas, tanto de los elementos extraídos, como de los cuerpos embebidos en hormigón, y se realiza el nuevo tablero, la nueva campana y el nuevo accionamiento, incluso dándole al tablero las formas curvas que muchas veces tienen.

Un punto delicado es la reposición de las bandas de cierre en el cuerpo fijo y la reposición de los cierres inferiores con metal Babbit, que han de ser realizados por personal altamente cualificado y con mucha experiencia.

Los resultados obtenidos con esta solución dan garantía, tanto como cierre de seguridad, como de estanqueidad, dentro de los parámetros habituales en compuertas fabricadas ex novo.

3.4 *Cambio sólo de accionamientos*

De forma muy frecuente, la fuente de los problemas de una compuerta se encuentra en su accionamiento, estando tanto su cuerpo como su tablero y los cierres en buen estado de conservación.

Cuando esto ocurre, un cambio de accionamiento es lo más directo y sencillo. Esto es aplicable a cambios de cilindros de accionamiento oleohidráulicos antiguos de baja presión por otros nuevos de accionamiento de alta presión, con sus correspondientes bridas de adaptación a la campana existente. También en mecanismos accionados por husillos a los que, por ejemplo, se les puede adaptar un mecanismo tipo AUMA o similar.

Son viables actuaciones intermedias en las que, por ejemplo, se modifica la campana existente para permitir un cambio de estopadas en carga Esta operación no es posible en la compuerta de aguas arriba sin obturar antes el conducto.

Actuaciones más complejas permiten cambiar el mecanismo de la compuerta de aguas arriba, aun estando en carga, bien sea sustituyendo camisa y pistón de un accionamiento oleohidráulico o superponiendo un nuevo cilindro al vástago del existente.

3.5 *Uso de la compuerta de guarda como escudo*

Se encuentran bastantes casos en los que no existe compuerta de paramento, no se puede colocar un escudo y el estado de las compuertas existentes exige su reparación completa. Un caso muy normal es una galería de entrada hasta la compuerta de seguridad de gran longitud (cientos de metros) parcialmente aterrada que impide el acceso de submarinistas con seguridad suficiente o, simplemente, una embocadura muy irregular o demasiado grande que impide la solución con un escudo.

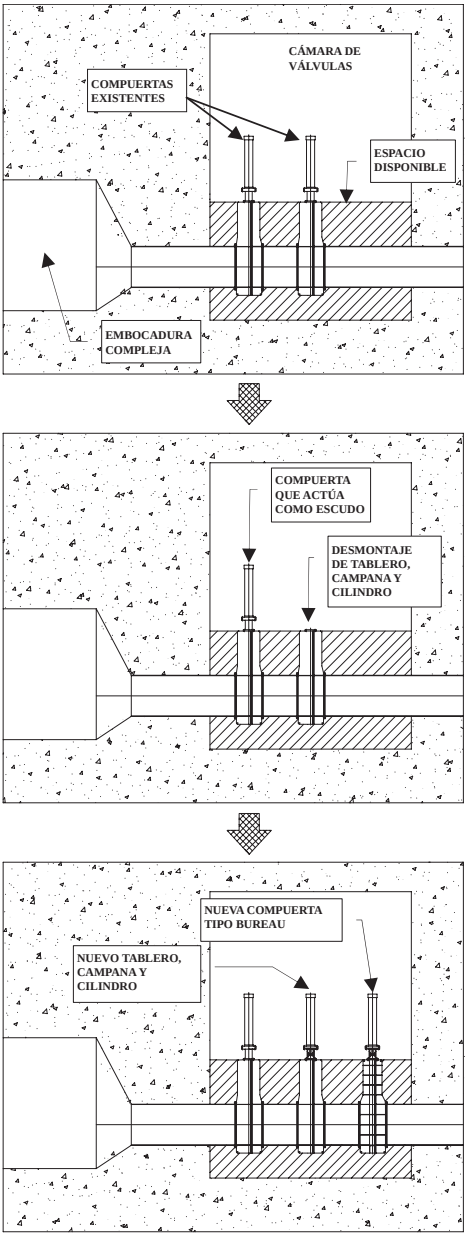
En estos casos una solución que estamos aplicando con éxito consiste en utilizar la compuerta de aguas arriba como escudo. Para ello, se cierra esta compuerta y se procede a retacarla mediante equipos de submarinistas. Se actúa igualmente en los conductos de by-pass si fuera necesario.

La válvula de aguas abajo se reparará de acuerdo con alguno de los sistemas anteriormente descritos y se coloca una tercera compuerta nueva, aguas abajo.

Así la compuerta antigua de seguridad queda como una simple ataguía para mantenimiento,

habitualmente condenada de forma que no la manipule el personal de la presa, sino que se use sólo para casos de reparación excepcional. La antigua compuerta de regulación pasa a ser la de seguridad y la nueva queda como de regulación.

Dependiendo de las distintas situaciones y necesidades de los desagües lo normal es optar por elementos de seguridad (compuerta Bureau) o por elementos de regulación (compuertas Taintor de fondo o válvulas Howell-Bunger).



• Tercera compuerta Bureau

Se emplea esta solución en aquellas ocasiones en que no se necesita regular caudales o bien cuando el espacio disponible en sentido longitudinal es escaso, incluso aunque se hayan de regular caudales.

Un desarrollo longitudinal típico para un conducto de 1,5 m serían 3 o 4 metros.

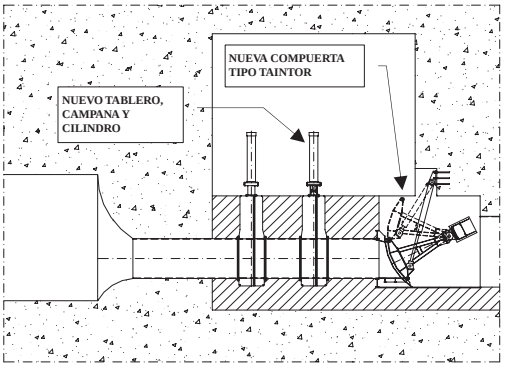
La principal ventaja es la robustez y garantía de la compuerta tipo Bureau y el inconveniente, que esta compuerta no está diseñada para regular, si bien esto no quiere decir que no sea capaz de hacerlo. Es una solución que no exige que la compuerta esté en punta del conducto como ocurre con las Taintor y las Howell-Bunger. Han de estudiarse cuidadosamente las demandas de aire y la durabilidad del tablero.

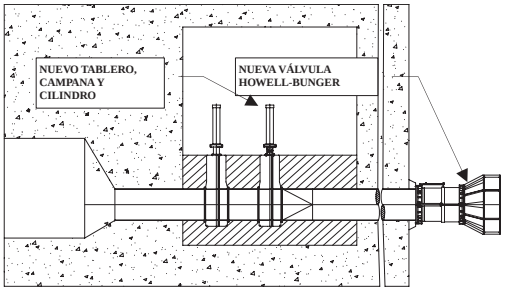
• Tercera compuerta Taintor de fondo

Cuando se necesita regular caudales en una amplia gama de los mismos, la solución con compuerta Taintor es perfecta. Esta compuerta se debe situar como fin de conducto y preferentemente en zonas con buen acceso a la atmósfera para saciar su demanda de aire. No presenta obstrucciones al flujo cuando está completamente abierta y es estable en regulación. Requiere más espacio en planta y más desarrollo en altura que otras compuertas pero es habitual incorporarlas a la cámara existente cuando la galería de salida es amplia, realizando unos picados de hormigón relativamente moderados o bien situarlas a la salida del conducto fuera del cuerpo de presa colocándolas en un edificio adosado al paramento.

• Tercera Válvula Howell-Bunger

Si se necesita regular caudales y a la vez disipar la energía del agua, las válvulas Howell-Bunger son muy adecuadas. Simples, robustas y económicas permiten regular caudal sin ningún problema de vibraciones, aireación o accionamientos complicados.





El problema que plantean es que presentan obstrucciones al flujo de agua por lo que se recomiendan sólo cuando haya rejas en la embocadura o cuando se considere que la cota de toma o la configuración del vaso del embalse es tal que no se esperan troncos, palos, etc.

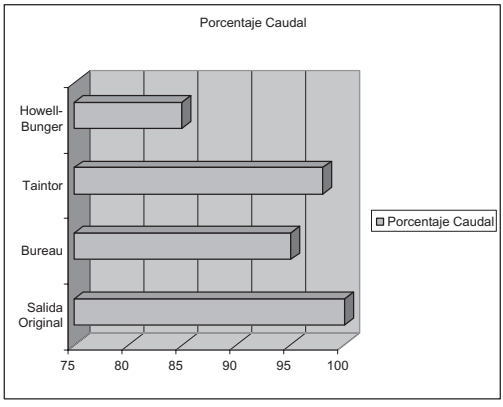
El chorro disperso que produce la Howell-Bunger es a veces su propio enemigo puesto que puede afectar a laderas, a alguna subestación eléctrica cercana, …

En estos casos, la válvula que nació para dispersar el chorro se pervierte a si misma y se convierte, mediante un concentrador, en una válvula que produce un chorro cilíndrico que necesita en su caída un cuenco amortiguador.

Este concentrador ha de ser cuidadosamente diseñado para evitar el lanzamiento de agua hacia atrás y las vibraciones, que podrían llegar a destruir la válvula.

4 AFECCIÓN A LOS CAUDALES DESAGUADOS DE CADA SOLUCIÓN

Una consideración a realizar es la afección a los caudales desaguados. Si consideramos que se han mantenido los blindajes de los conductos en su misma sección, se producen afecciones a los caudales al sustituir válvulas de compuerta o mariposas por compuertas tipo Bureau, fundamentalmente



debido al uso de transiciones de sección circular a rectangular.

En el caso de las válvulas Howell-Bunger se puede llegar a perder en torno a un 15% del caudal original debido a su concepción como elemento disipador de energía.

La Taintor de fondo, al ser de sección de paso completa y libre a plena apertura no presenta en si misma reducción de caudales salvo que se necesite una transición de sección circular a rectangular.

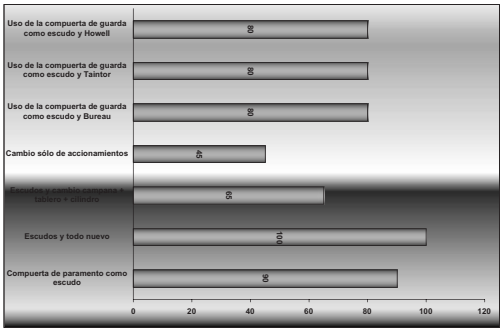
La estimación de la tabla anterior se ha hecho con respecto a un conducto original libre. Si consideramos, por ejemplo, un conducto previo compuesto por una mariposa y una válvula Larner-Johnson, que se sustituye por una Bureau y una Taintor, nos encontraremos con que hemos aumentado la capacidad de desagüe del conducto, al haberle reducido las pérdidas de carga.

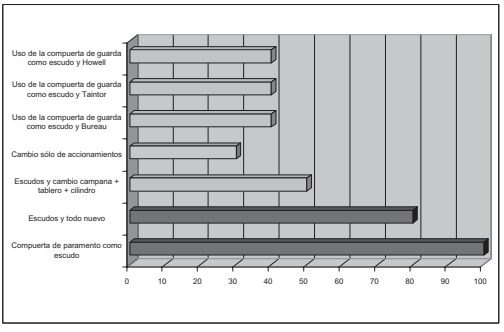
5 IMPACTO ECONÓMICO DE CADA SOLUCIÓN

En el siguiente gráfico, se muestra el impacto económico de cada solución en términos relativos. Debe tenerse en cuenta que en la realidad la variación de condiciones hace prácticamente única cada actuación pero en líneas generales esta tabla puede ser representativa.

Obviamente, la solución de cambiar sólo los accionamientos es la más económica, pero no se ataca el estado de los cierres de los elementos ni el estado de conservación de los mismos.

A continuación está la solución de conservar los cuerpos de los elementos existentes, que si bien soluciona los problemas de accionamiento, de estanqueidades y del elemento de obturación en si, deja los cuerpos de válvula existentes. Cuando se está muy seguro del estado de conservación de los mismos esta solución es muy adecuada. También lo es cuando el coste de retirar estos cuerpos es





elevado (picado de hormigones, extracción de los voluminosos y pesados cuerpos).

En un punto intermedio están las soluciones de colocar una línea con una tercera compuerta. En muchos casos aparece como una solución equilibrada ya que no es tan costosa como la sustitución total pero si deja una compuerta reparada, una completamente nueva (capaz incluso de regular caudales si se ha elegido el elemento adecuado) e incluso la de aguas arriba para poder obturar el conducto y realizar labores de mantenimiento.

Las más costosas son las de sustitución total, dependiendo si existe elemento de obturación o se necesita uno nuevo.

6 RIESGO SEGURIDAD EJECUCIÓN DE CADA SOLUCIÓN

Igualmente que con el caso del impacto económico, el riesgo constructivo, no en términos de seguridad personal, sino de seguridad de la instalación durante la realización de las obras, prácticamente depende de cada presa en concreto. No obstante la tabla que se adjunta puede servir para dar una idea inicial de la cuestión.

Si bien la solución en verde es la de menor riesgo, el alcance de la reparación es muy limitado.

El equilibrio entre riesgo y economía, puede alcanzarse en las soluciones representadas en azul claro.

La solución de cambio sólo de accionamientos es la de menor riesgo. Sin embargo se ha de ser muy cuidadoso al actuar sobre el accionamiento del elemento de aguas arriba si no se dispone de obturación en el conducto.

Las soluciones con escudo implican riesgo, en tanto en cuanto el escudo pudiera fallar o el apoyo del mismo.

Se consideran de mayor riesgo las soluciones en que todo se desmonta.

Las soluciones, actuando con la compuerta de paramento existente se han considerado incluso de mayor riesgo que las de colocación de escudo, dado que en ocasiones se desconoce el estado de la compuerta de paramento y los criterios de su diseño, mientras que en el caso de escudo no hay estas incertidumbres.

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Los aliviaderos tipo laberinto. Un nuevo enfoque para su cálculo hidráulico

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ABSTRACT: At present, The Polythencic University of Madrid, the *CEDEX* and *JESÚS GRANELL Ingenieros Consultores* Company are carrying out an investigation project on labyrinth spillways. As a part of this project, it is proposed a new method for the hydraulic calculation of this type of spillways, which is based on the main facts occurred: the obliquity of the diagonal faces of the weir referred to the direction of the flow upstream in the approximation, and the nappe interference. The method is going to be improved and validated by physical and numerical modelling as a part of the investigation project. However, the comparison of its preliminary results with other methods, and also with some physical models available provides encouraging results.

RESUMEN: En la actualidad la Universidad Politécnica de Madrid, el Centro de Experimentación de Obras Públicas (CEDEX) y la empresa JESÚS GRANELL Ingenieros Consultores, están llevando a cabo un proyecto de investigación en materia de aliviaderos tipo laberinto. Dentro de este proyecto se propone un nuevo método para el cálculo hidráulico de estos aliviaderos de tipo teórico—experimental de formulación sencilla y basado en los principales fenómenos que condicionan la capacidad hidráulica de estas estructuras: la oblicuidad de las caras diagonales del vertedero respecto de la dirección del flujo en la alimentación y la interferencia de láminas. El procedimiento va a ser mejorado y validado, dentro del proyecto de investigación comentado, mediante modelación física y numérica. No obstante, el contraste de sus resultados con otros procedimientos existentes y con los resultados de algunas modelaciones físicas disponibles ofrece resultados esperanzadores.

1 INTRODUCCIÓN

En la actualidad la Universidad Politécnica de Madrid, el Centro de Estudios Hidrográficos del CEDEX y la empresa JESÚS GRANELL Ingenieros Consultores están llevando a cabo un proyecto de investigación subvencionado por el Ministerio de Medio Ambiente y Medio Rural y Marino sobre la mejora de la capacidad de desagüe en presas existentes.

Dentro del referido proyecto de investigación, se encuentran los aliviaderos tipo laberinto que, como se conoce, pueden ofrecer una elevada capacidad hidráulica para sobre-elevaciones reducidas.

Clásicamente los aliviaderos tipo laberinto se calculan hidráulicamente mediante procedimientos realizados a partir de la extrapolación de estudios experimentales, hasta obtener unos ábacos o formulaciones que se aplican de manera genérica a cualquier geometría de aliviadero tipo laberinto.

Tales procedimientos, si bien constituyen una buena aproximación, son insuficientes exigiendo en muchos de los casos la modelación física del aliviadero a escala reducida. En la actualidad se ha abierto el campo de la modelación numérica de estructuras hidráulicas sin que haya llegado a reemplazar a la modelación física.

Se ha elaborado un procedimiento de cálculo, de tipo teórico—experimental y elevada sencillez tanto a nivel conceptual como en su aplicación, que se basa en el análisis de los principales fenómenos que afectan al comportamiento de un aliviadero tipo laberinto: oblicuidad del vertedero (en las caras de dirección diagonal) respecto del flujo en la alimentación, y la interferencia de láminas vertidas por las caras contiguas del vertedero.

Este procedimiento, que está siendo validado mediante un elevado número ensayos físicos así como mediante modelación numérica por el Método de los Elementos Finitos y las

Partículas (PFEM) dentro del referido proyecto de investigación, a día de hoy ofrece buenos resultados y se considera que puede contribuir a un mejor diseño y comprobación hidráulica de este tipo de estructuras.

2 MÉTODOS ACTUALES PARA EL CÁLCULO HIDRÁULICO DE UN ALIVIADERO TIPO LABERINTO

Existen tres maneras de analizar la hidráulica de un aliviadero tipo laberinto:

1. Mediante procedimientos experimentales de cálculo cuyos resultados se extrapolan a una determinada gama de geometrías y rangos de funcionamiento.
2. Mediante ensayos hidráulicos sobre modelaciones físicas a escala reducida.
3. Mediante modelaciones numéricas por el MEF.

Actualmente, el diseño inicial de los aliviaderos tipo laberinto se realiza por el primero de los tres medios comentados, diseño este que exige, generalmente, ser comprobado por una modelación física. La modelación numérica, que a día de hoy sigue exigiendo un importante costo en tiempo y recursos, permite realizar ajustes adecuados sobre los diseños iniciales, pero no ha eliminado, actualmente, la necesidad de construir modelos físicos a escala reducida.

Los procedimientos experimentales permiten el diseño y cálculo hidráulico de los vertederos tipo laberinto con bastante facilidad, mediante la aplicación de ábacos o ecuaciones sobre la geometría en cuestión. Los más comúnmente empleados son los siguientes:

1. El elaborado por A. Pinto de Magalhäes en 1983.
2. El elaborado porFrederick Lux III and David L. Hinchliff en 1985.
3. El elaborado por J. Paul Tullis, Nosratollah Amanian and David Waldron en marzo de 1995.

3 UN NUEVO PROCEDIMIENTO PARA EL CÁLCULO HIDRÁULICO DE ALIVIADEROS TIPO LABERINTO

3.1 *Generalidades*

La particularidad de los aliviaderos tipo laberinto frente a los aliviaderos convencionales reside en la forma del vertedero, que es asimilable a un "zig—zag". Esta especial morfología incrementa la longitud de vertedero "a encajar" en un determinado

Figura 1. Aliviadero tipo laberinto en el embalse de Puentes (Murcia).

"espacio disponible", con el fin de aumentar la capacidad de evacuación respecto a la de un vertedero recto convencional ubicado en el mismo espacio y para igual carga hidráulica.

Esta supuesta ventaja—mayor capacidad hidráulica para iguales condiciones de espacio disponible y carga hidráulica—se hace incierta cuando la carga hidráulica sobre el umbral del vertedero crece por encima de determinados valores.

Un vertedero tipo laberinto se construye a partir de la disposición de manera repetida, de módulos iguales a lo largo de una directriz geométrica. De esta manera se define a partir del número de módulos que se adosan y la definición genérica del módulo.

Se definen, por tanto, los siguientes parámetros geométricos:

a: Semilongitud de muros longitudinales (m)
b: Longitud de muros diagonales (m)
m: Pendiente del canal aguas abajo del vertedero
$l = 4a + 2b$: Longitud desarrollada del módulo (m)
w: Longitud recta o proyectada del módulo (m)
y: Distancia entre vértices del módulo media en la dirección ortogonal a la directriz (m)
α: Ángulo de los muros diagonales con el eje del módulo
α_{max}: Ángulo correspondiente al módulo triangular de misma *"w"* y mismo *"y"*
n: Número de módulos
p: Altura del vertedero en su parte de aguas arriba (m)
d: Altura del vertedero en su parte de aguas abajo (m)
t: Espesor de los muros (m)
$W = nw$. Longitud total recta o proyectada del vertedero en laberinto (m)
$L = nl$. Longitud total desarrollada del laberinto (m)

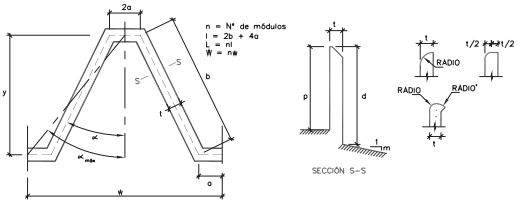

Figura 2. Aliviadero tipo laberinto. Definición geométrica.

Hay además una serie de parámetros adimensionales que permiten caracterizar la geometría:

l/*w*: Amplificación de longitud
w/*p*: Relación de aspecto vertical
h/*p*: Relación carga-altura o carga hidráulica relativa. Siendo h la carga hidráulica, por aguas arriba, medida desde el umbral.

Si se analiza con detenimiento la forma de los aliviaderos tipo laberinto y se relaciona con lo que se conoce de su comportamiento hidráulico, es posible establecer que existen dos tipos de fenómenos que condicionan su funcionamiento[1], y que lo distinguen de los aliviaderos convencionales:

– La oblicuidad de las caras diagonales del vertedero respecto de la dirección del flujo en la alimentación
– La interferencia de láminas vertientes por las caras adyacentes de cada módulo del vertedero

El nuevo procedimiento para el cálculo de aliviaderos tipo laberinto que se propone, parte del análisis de estos dos fenómenos: el primero se analiza, de una forma teórica, mediante la aplicación del teorema de la cantidad de movimiento, mientras que el segundo se analiza a partir de investigaciones experimentales pretéritas.

3.2 *La oblicuidad del vertedero*

El análisis del comportamiento hidráulico de un vertedero, situado en un canal, y oblicuo a la dirección del flujo aguas arriba a partir de la aplicación del teorema de la cantidad de movimiento, es analizado con detenimiento por A. Osuna. Su desarrollo matemático se ha adaptado para ser aplicado al caso de aliviaderos tipo laberinto y se adjunta a continuación (A. Osuna, 2000. "Determinación teórica del desagüe en un vertedero oblicuo").

El teorema de la cantidad de movimiento establece que la fuerza que se ejerce sobre una porción de fluido en movimiento permanente es igual al flujo de la cantidad de movimiento en su contorno. Se ha procedido a aplicar el teorema sobre un fluido perfecto que discurre por una canal y en el que se ubica un vertedero oblicuo.

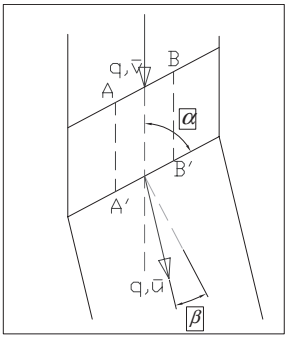

Figura 3. Vertedero oblicuo en un canal.

Si se selecciona la porción de fluido AA' y B'B, siendo la distancia AB la unidad (para hablar en términos de caudales unitarios), se obtiene que la cantidad de movimiento pasa de $pq\bar{v}$ en AB a $pq\bar{u}$ en A'B', siendo:

ρ: peso específico del agua (t/m^3)
q: caudal unitario (m^3/s/m)
$\bar{v}$: vector velocidad en AB (m/s).
$\bar{u}$: vector velocidad en A'B' (m/s)
β: ángulo que forman las líneas de corriente a la salida del vertedero con la perpendicular al mismo.

Por ser el flujo de la cantidad de movimiento igual a la resultante de las fuerzas exteriores actuantes sobre la porción, la proyección de este flujo sobre el plano del vertedero debe ser nula:

$$pq\bar{v}cos\alpha - pq\bar{u}sen\beta = 0 \qquad (1)$$

que es equivalente a:

$$v = \frac{usen\beta}{\cos\alpha} \qquad (2)$$

El caudal vertido por unidad de longitud de vertedero:

$$q_a = \bar{u}h_c\cos\beta \qquad (3)$$

En donde h_c es el espesor de lámina contraída sobre el vertedero y que puede tomarse 0,667h.

El caudal por unidad de anchura de canal:

$$q = v(p+h) = \frac{q_a}{\cos(90-\alpha)} = \frac{\bar{u}h_c\cos\beta}{\cos(90-\alpha)} \qquad (4)$$

[1]Dejando al margen el posible anegamiento por aguas abajo del vertedero y que es un fenómeno común al conjunto de los aliviaderos.

Sustituyendo el valor de v de (1) en (3):

$$tg\beta = \frac{h_c}{p+h} tg(90-\alpha) \quad (5)$$

Que permite calcular el caudal por unidad de longitud de vertedero:

$$q_a = \frac{2}{3} C_d \sqrt{2g} h^{3/2} \cos\beta \quad (6)$$

El resultado que se obtiene es que la dirección del flujo a la salida del vertedero es una intermedia entre la perpendicular al paramento del vertedero y la del flujo en el canal aguas arriba del mismo. Esta dirección depende de la relación entre el espesor de lámina contraída sobre el vertedero h_c y el calado total aguas arriba del vertedero $p + h$ en un punto suficientemente alejado. De esta manera:

- Cuando esta relación $h_c/(p+h)$ es muy pequeña, el flujo a la salida del vertedero adopta una dirección cercana a la perpendicular al paramento del mismo. El caudal total desaguado se maximiza con relación al que desaguaría un vertedero normal al canal.
- Para valores crecientes de la relación $h_c/(p+h)$ el flujo a la salida del vertedero adopta una dirección que se acerca, cada vez más, a la del canal y el caudal total desaguado se va acercando al que desaguaría un vertedero normal a la dirección del flujo aguas arriba del mismo.

Las expresiones obtenidas se pueden aplicar a un aliviadero en laberinto, sin más que integrar las ecuaciones a lo largo de la directriz de uno de los módulos y variando el ángulo β.

La aplicación de esta formulación en vertederos en laberinto con importantes desarrollos, como es usual en este tipo de estructuras, establece que el funcionamiento hidráulico se ve fuertemente condicionado por la oblicuidad de los mismos con relación a la dirección del flujo.

3.3 *La interferencia de láminas*

Este fenómeno se produce por el impacto de las láminas vertientes en la zona próxima a los vértices de aguas arriba del vertedero. Estos impactos generan pérdidas de carga que reducen la capacidad hidráulica del vertedero.

Indlekofer y Rouvé, analizan el fenómeno de interferencia de láminas en vertederos poligonales y el efecto que esta interferencia tiene en la capacidad hidráulica del vertedero (Horst Indlekofer and Gerhard Rouvé, 1975. "Discharge over poligonal weirs").

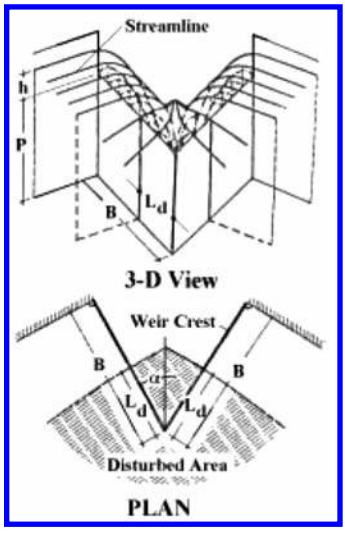

Figura 4. Indlekofer y Rouve, 1975. Interferencia de láminas.

Se presenta una figura de la investigación referida.

Ellos definen una serie de parámetros:

- l_d: longitud de interferencia de láminas o zona de cada pared del vertedero que se ve afectada por la interferencia de láminas.
- h: carga hidráulica sobre el vertedero

En la figura mostrada C_m es un coeficiente que mide la pérdida de capacidad en la zona de interferencia de láminas l_d que se denomina coeficiente de interferencia de láminas y μ es el coeficiente de desagüe, equivalente a C_d.

De este modo, el caudal evacuado por cada una de las caras del vertedero será:

$$Q = \frac{2}{3} \mu \sqrt{2g} h^{1,5} \left(b - (1 - C_m) l_d \right) \quad (7)$$

$$l_{de} = (1 - C_m) l_d \quad (8)$$

$$Q = \frac{2}{3} \mu \sqrt{2g} h^{1,5} (b - l_{de}) \quad (9)$$

En donde $b - l_{de}$ representa la longitud efectiva del vertedero[2].

De la referida investigación se obtiene una relación lineal, a escala logarítmica, como la que se muestra:

[2] Se ha cambiado la nomenclatura del parámetro L_{de} de Indlekofer y Rouvé, 1975, por l_{de} para que toda la notación sea coherente.

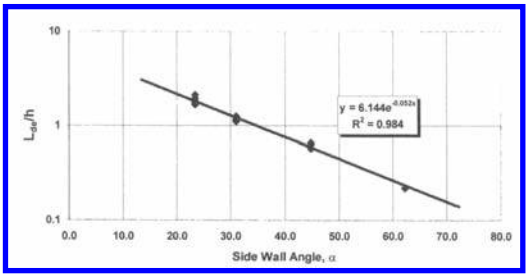

Figura 5. Indlekofer y Rouve, 1975. Relación $L_{de} - \alpha$.

De tal modo que

$$\frac{l_{de}}{h} = 6.1e^{-0,052\alpha} \tag{10}$$

Expresión que es de aplicación siempre que:

$$\frac{l_{de}}{b} \leq 0,3 \tag{11}$$

Y de la que resulta:

$$Q = \frac{2}{3}\mu\sqrt{2g}h^{1,5}(2b - 12,2he^{-0,052\alpha}) \tag{12}$$

4 FORMULACIÓN DEL PROCEDIMIENTO

La ecuación (12) permite la obtención de la capacidad hidráulica en un vertedero poligonal con la alimentación ortogonal a las paredes del vertedero. No obstante la alimentación en vertederos tipo laberinto tiene la dirección del canal, por lo que para aplicarla a estas estructuras hidráulicas exige una corrección.

En efecto, tal y como se muestra en la figura, la interferencia de láminas en el caso de los ensayos de Indlekofer y Rouvé se produce con las líneas de corriente perpendiculares a las paredes del mismo, mientras que en el caso que nos ocupa al ser la alimentación ortogonal a la directriz del laberinto, es preciso sustituir el ángulo 2α por el equivalente 2γ que resulta de aplicar el teorema del flujo de la cantidad de movimiento.

$$\gamma = \alpha + \beta(h,\alpha,p) \tag{13}$$

β se calcula mediante la expresión (5):

La expresión final que permitirá obtener la capacidad hidráulica es:

$$Q = \frac{2}{3}\mu\sqrt{2g}h^{1,5}\left[2b\cos\beta - 2l'_{de}\right] \tag{14}$$

$$\frac{l'_{de}}{h} = 12,2e^{-0,052(\alpha+\beta)} \tag{15}$$

De tal modo que si se sustituyen los diversos parámetros se obtiene:

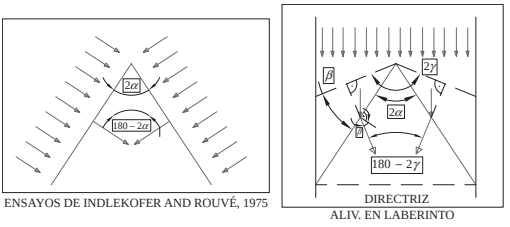

Figura 6. Interferencia de láminas en vertederos en laberinto.

$$Q = \frac{4}{3}\mu\sqrt{2g}h^{1,5}\left[b\cos\beta - 6,1he^{-0,052(\alpha+\beta)}\right] \tag{16}$$

Esta ecuación se puede aplicar para el cálculo de la capacidad hidráulica de un vertedero tipo laberinto con módulos triangulares ($\alpha = 0$).

No obstante, los aliviaderos tipo laberinto suelen tener formas trapeciales, por lo que se ha realizado una simplificación que consiste en calcular la pérdida de capacidad derivada de la interferencia de láminas para un vertedero triangular equivalente, esto es, aquel con la misma w, pero con $\alpha = \alpha_{max}$:

La longitud de interferencia de láminas efectiva será:

$$l_{de} = 6.1he^{-0,052(\alpha_{máx}+\beta_{máx})} \tag{17}$$

De manera que (16) se convierte en:

$$Q = \frac{4}{3}\mu\sqrt{2g}h^{1,5}\left[b\cos\beta - 6,1he^{-0,052(\alpha_{max}+\beta_{max})}\right] \tag{18}$$

$$\beta_{máx} = atg\left(\frac{0,667h}{p+h}tg(90 - \alpha_{máx})\right) \tag{19}$$

$$\alpha_{máx} = atg\left(\frac{0,5wtg\alpha}{0,5w - 2a}\right) \tag{20}$$

$$\beta = atg\left(\frac{0,667h}{p+h}tg(90 - \alpha)\right) \tag{5}$$

El factor μ es el coeficiente de desagüe de un vertedero en pared delgada que puede tomarse de la conocida expresión experimental:

$$\mu = 0,611 + 0,075\frac{h}{p} \tag{21}$$

La ecuación (18) es aplicable a cualquier geometría de vertedero tipo laberinto.

Hasta el momento, el procedimiento propuesto se ha aplicado a tres geometrías distintas de vertederos tipo laberinto y que han sido ensayadas

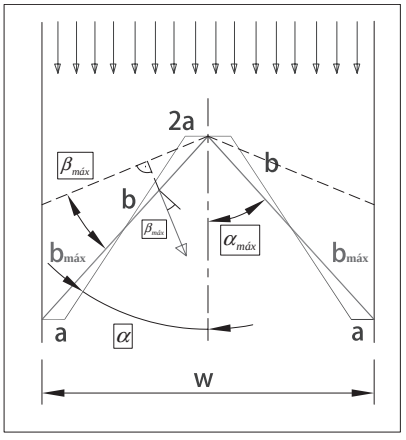

Figura 7. Interferencia de láminas. Simplificación para vertederos trapeciales.

hidráulicamente en modelos a escala reducida. Las desviaciones obtenidas han sido de menos del 10%.

Además, se han contrastado los resultados con los de otros dos procedimientos de cálculo existentes (Pinto de Magalhäes, 1983 y Tullis, Amanian y Waldron, 1995) obteniéndose diferencias, en general, inferiores al 10%.

5 CONCLUSIÓN

En la actualidad se está llevando a cabo una investigación sobre aliviaderos tipo laberinto dentro del proyecto de investigación "Mejora de la capacidad de desagüe de presas existentes mediante aliviaderos en sifón y aliviaderos en laberinto" que se desarrolla con las ayudas relativas a los proyectos I+D+i correspondientes al subprograma de medioambiente y de ecoinnovación del Ministerio de Medio Ambiente y que está desarrollando, de manera conjunta la U.P.M., el CEDEX y la empresa JESÚS GRANELL Ingenieros Consultores.

Como parte de esta investigación se ha elaborado un procedimiento para el cálculo hidráulico de aliviaderos tipo laberinto, que se basa en los dos principales fenómenos que condicionan el funcionamiento de este tipo de aliviaderos: la oblicuidad de las caras diagonales respecto de la dirección de la alimentación hidráulica y la interferencia de láminas existente en la zona de los vértices de aguas arriba.

La aproximación al primero de los fenómenos se ha realizado, de manera teórica, mediante la aplicación del teorema de la cantidad de movimiento, mientras que la aproximación al segundo se ha realizado a partir de la formulación obtenida de un estudio experimental sobre vertederos poligonales y mediante una simplificación del vertedero en laberinto trapecial a uno triangular equivalente.

Con ello se ha obtenido una formulación consistente en una única ecuación, que permite el cálculo hidráulico de cualquier vertedero tipo laberinto.

El procedimiento se ha comprobado contrastándolo con los resultados de tres modelos físicos relativos a tres vertederos tipo laberintos existentes. Se han obtenido desviaciones inferiores al 10%.

Asimismo, se han contrastado los resultados del procedimiento propuesto con los de otros dos procedimientos (Pinto de Magalhäes, 1983 y Tullis, Amanian y Waldron, 1995). Se han obtenido diferencias, en general, inferiores al 10%.

Se considera que es posible mejorar el procedimiento propuesto especialmente en lo relativo al fenómeno de interferencia de láminas. Para ello y para poder validar el procedimiento se va a realizar una campaña de ensayos físicos sobre modelos a escala reducida, en los que además se van a analizar otros fenómenos tales como las condiciones de alimentación al vertedero.

AGRADECIMIENTOS

Hay que agradecer al Ministerio de Medio Ambiente y Medio Rural y Marino las ayudas para la realización del proyecto "Mejora de la capacidad de desagüe de presas existentes mediante aliviaderos en sifón y aliviaderos en laberinto" desarrollado de manera conjunta por la U.P.M., el CEDEX y la empresa JESÚS GRANELL Ingenieros Consultores, y relativas a los proyectos I+D+i correspondientes al subprograma de medioambiente y de ecoinnovación y sin las que la presente investigación no habría sido posible.

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"Ejecución, en carga, de nueva toma intermedia por perforación mediante tuneladora. Presa de Oliana"

Gonzalo Rabasa Pérez
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RESUMEN: Los trabajos de construcción de una nueva toma en la presa de Oliana, en el río Segre en Cataluña, consistieron en una perforación del cuerpo de presa. Esta se comenzó en el paramento de aguas abajo y perforó horizontalmente la infraestructura hacia aguas arriba. Para realizarlo se utilizó una micro TBM (tuneladora) empujada por unos gatos hidráulicos.

Debido a la imposibilidad de vaciar el embalse, la operación se tuvo que realizar con una importante carga de agua en el extremo final de la perforación.

Para comenzar los trabajos se tuvo que construir un muro de reacción en el paramento aguas abajo. Allí se situó la micro TBM, que obtendría la fuerza necesaria de empuje contra el muro por medio de unos gatos hidráulicos.

Cuando estos gatos estaban completamente extendidos se recogían, momento en que colocaba un elemento de tubería en el alojamiento resultante. Esta tubería tenía una doble función: por un lado constituía la transmisión del esfuerzo de empuje desde los gatos hasta la TBM y por otro, constituiría posteriormente la conducción definitiva de la toma.

En el momento en que la tuneladora llegó al paramento en el lado del embalse, el agua del mismo penetró en el espacio en forma de corona cilíndrica que quedaba entre la perforación y la tubería, mientras que la TBM quedaba suspendida a la espera de su extracción por este mismo paramento. Una junta tórica, elemento clave en este sistema constructivo, proporcionaba la estanqueidad necesaria en el orificio anular existente en el arranque de la perforación.

Por ultimo, el espacio mencionado fue sellado mediante inyecciones y la TBM se extrajo por el paramento mojado por medio de una grúa situada sobre la presa. El interior de la conducción permaneció seco gracias a una escotilla existente en el extremo del tubo, la cual fue retirada del mismo modo que la TBM una vez colocadas en el paramento de aguas abajo las válvulas.

ABSTRACT: The works to make a new outlet in the Oliana Dam on Segre River, in Catalonia, consisted in boring the body of the dam. The boring was started at the dry side of the wall and was driven horizontally towards the upstream face. A TBM (tunnel boring machine) and the pipe jacking method were used.

Due to the impossibility to empty down the reservoir, this task were carried out with an important water pressure at the end of the boring.

The works started with the construction of a reaction wall at the dry side of the dam. The micro TBM was located there to bore the concrete of the dam by pushing hydraulic jacks against the reaction wall. In this way, the needed force for the boring was obtained.

When the jacks were fully expanded they were retrieved, so it could be laid the necessary length of pipe between the boring way and the jacks. This pipe had a double function: it transfered the effort force from the wall to the TBM to continue the boring operation and it was going to be the definitive outlet of the dam.

At the time the boring reached the reservoir, the water penetrated the cylindrical crown that was formed between the pipe and the boring, while the TBM stayed waiting for the extraction over the wet side of the dam. A toric joint at the beginning of the drilling hole stopped the water. This joint was a key importance element in this working system.

Finally, the cylindrical crown was grouted to make it impervious and the TBM was recovered by a crane located on the dam. The inside of the pipe remained hermetic thanks to a hatchway at the end of the pipe line, which was removed by the crane, after placing the valves at the dry side of the dam.

PRINCIPALES DATOS DE LA ACTUACION

- Cota perforación: 491 m.s.n.m.
- Cota embalse: 518 m.s.n.m.
- Diámetro de perforación: 1.295 mm
- Longitud de perforación: 17,200 ml
- Diámetro tubería: 1.000 mm/interior
- Diámetro tubería: 1.260 mm/exterior
- Tipo de tubería: Hormigón Armado con camisa de chapa machihembrada y junta de goma

1 INTRODUCCION

La presa de Oliana almacena aguas del rio Segre, siendo su máxima capacidad de embalse de 101 Hm^3, su cota de coronación la 520,04 y su nivel máximo de normal (N.M.N.) la cota 518,74.

Debido a diversos problemas de explotación que presentaba la citada presa, CONFEDERACIÓN HIDROGRÁFICA DEL EBRO adjudicó el pasado 29 de Abril a OBRAS Y SERVICIOS PÚBLICOS, S.A. (O.S.E.P.S.A.), empresa especializada en trabajos en presas y canales, las actuaciones de EMERGENCIA PARA LA INSTALACIÓN DE UNA TOMA INTERMEDIA EN LA PRESA DE OLIANA.

La citada actuación consta de dos partes diferenciadas. Por un lado la ejecución de una toma intermedia-desagüe de medio fondo en la cota 491,00 y por otro lado la remodelación de los órganos de alivio de la presa.

La ejecución de la toma intermedia-desagüe de medio fondo constará de una perforación de 17,20 ml de longitud en el cuerpo de la presa mediante hinca de tubería de hormigón armado con camisa de chapa de diámetro 1.000 mm. Esta perforación se ejecuta desde el talud de aguas abajo siendo la cota de agua en el momento de perforación la 518,00 resultando una columna de agua de alrededor de 27 ml, siendo la peculiaridad de la actuación que el mismo equipo de perforación funciona como tapón. Esta operación que dota a la actuación de gran rapidez y reducción de costes es pionera en Europa.

Esta conducción, tras la caseta de válvulas situada en el talud de la presa, continuará con una conducción de acero inoxidable de 35 ml y diámetro 1.000 mm hasta llegar al pie del talud de la presa, llegados al terreno la tubería será de acero al carbono que continuará 100 ml aguas abajo, donde se bifurcará, partiendo desde allí la tubería que dará riego a las tierras de Oliana, Basella y Peramola La otra rama de la bifurcación, que vierte las aguas al rio, queda a disposición de la explotación de la presa para ser utilizada como desagüe de medio fondo para casos de emergencia.

La siguiente explicación corresponde en concreto a los trabajos de PERFORACION DE LA PRESA.

2 DESCRIPCION DE LA ACTUACION

2.1 *Fase nº 1: Demolición para la Creación Zona de trabajo*

En primer lugar se procedió a la preparación de una plataforma de trabajo que posteriormente alojaría la compuerta de válvulas de seguridad. Para crear esta plataforma se procedió a la demolición de un dado de hormigón de dimensiones en planta 4,00 × 4,80 ml. Esta demolición se ejecutó mediante la combinación de distintos equipos especiales (corte con Hilo de Diamante + Gatos Hidráulicos + Dardas). La demolición, para facilitar su posterior extracción desde el cuerpo de la presa, que debía hacerse con grúas de gran tonelaje por la situación del punto de actuación en el centro del cuerpo de la presa, debía resultar en grandes bloques de hormigón. Los bloques que resultaron tenían un volumen medio aproximado de 1 m^3. El proceso puede observarse en la Figura nº 1.

2.2 *Fase nº 2: Muro de Empuje*

Finalizados los trabajos de creación de la plataforma de trabajo, se procedió a la ejecución del muro que debía soportar el empuje de bastidor que a su vez empujaba el conjunto de perforación. Este muro se dimensiono para soportar un empuje de hasta 200 Tn/m^2. Las características de este muro resultaron ser las siguientes:

Dimensiones 1,00 × 4,00 en planta y 2,00 ml de altura. (Ver Figura n° 2)

Hormigón HA-35 armado con doble parrilla Ø32 vertical—Ø16 horizontal.

Figura 1.

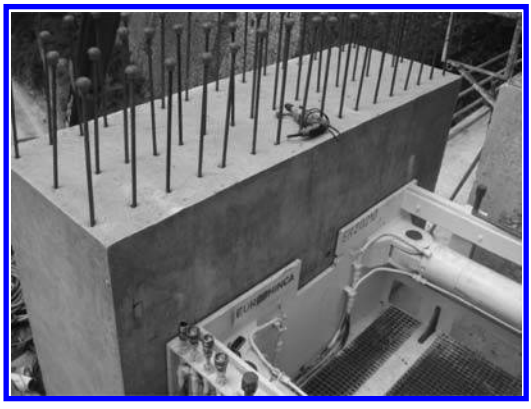

Figura 2.

Anclaje a hormigón existente mediante Ø50 L = 1 + 1 ml cada 20 cm en tres líneas al tresbolillo.

2.3 *Fase nº 3: Pieza de Ataque Estanca con Doble Junta de Estanqueidad*

La pieza de ataque estanca consiste en tres aros metálicos sobrepuestos longitudinalmente en el sentido de la perforación, unidos mediante tornillería. Entre cada uno de dos aros esta alojada una junta contra la que deslizara el tubo hincado y que evitara el escape el agua que rellenara el espacio del sobre corte. (Ver Figura n° 3)

Esta pieza de ataque a su vez se atornillo a un bastidor metálico. Para el alojamiento de este bastidor metálico se ejecutó un muro armado de 0,50 ml de espesor armado con parrilla Ø16 # 20 y atado al hormigón de la presa con Ø16 L = 0,5 + 0,25 ml cada 0,5 ml al tresbolillo. El citado bastidor metálico presentaba Ø16 L = 0,25 ml cada 0,5 ml al tresbolillo. Todo esto aseguró la perfecta continuidad del conjunto bastidor metálico—hormigón muro—hormigón presa lo que evitó la aparición de filtraciones por esta zona. En esta parte de la preparación de la perforación, como se observa en la Figura n° 4, se tuvo especial cuidado, puesto que fue por este punto por donde se produjeron importantes problemas en una actuación similar realizada anteriormente.

Seguidamente se procedió a colocar la pieza de ataque sobre el citado bastidor, estando en este momento todo preparado para iniciar la instalación de los equipos de perforación.

2.4 *Fase nº 4: Perforación e Hincado de Tubería*

El **equipo de perforación** consta de los siguientes elementos:

- Perforadora hidráulica refrigerada por agua, con un diámetro de 1.260 mm, resultando una perforación de diámetro 1295 mm aproximadamente.
- Tapón estanqueidad (también denominado pieza tapón) interior tubería que avanza solidario tras la perforadora.
- Tubo con extremo "macho" en acero inoxidable para posterior acoplamiento de pieza de embocadura. Este tubo será el que avanza en primer lugar, tras la pieza tapón.
- Tubo de hormigón armado PN6 Ø interior 1.000 mm con camisa de chapa y junta de goma machihembrada.
- Bastidor para soporte de cilindros oleo hidráulicos para movimiento de aro de empuje. (ver figura n° 6)
- Decantador de detritus para reutilización de agua para refrigeración de perforadora. Incluido todo el equipo de bombeo de agua a la perforadora y extracción de lodo hacia el decantador.
- Equipo integrado de control formado por equipo informático, equipo laser de control de avance y equipo hidráulico.

Finalizados todos los preparativos se inicio la distribución de los citados equipos.

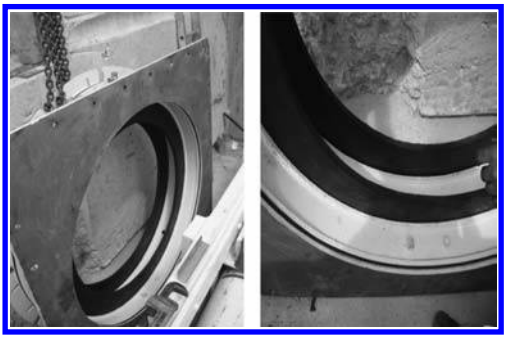

Figura 3.

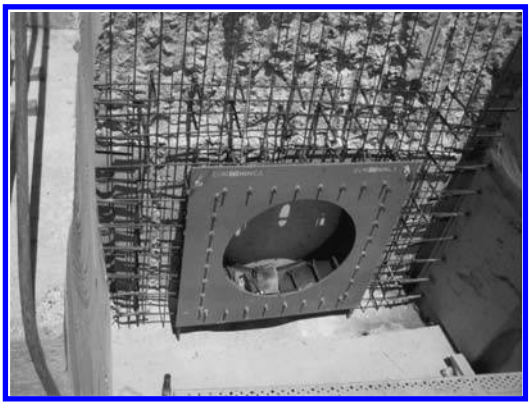

Figura 4.

Figura 5.

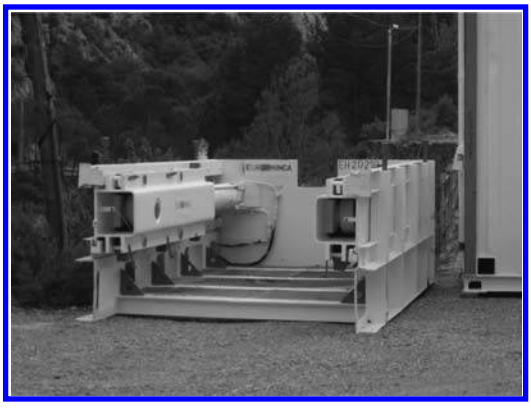

Figura 6.

Figura 7.

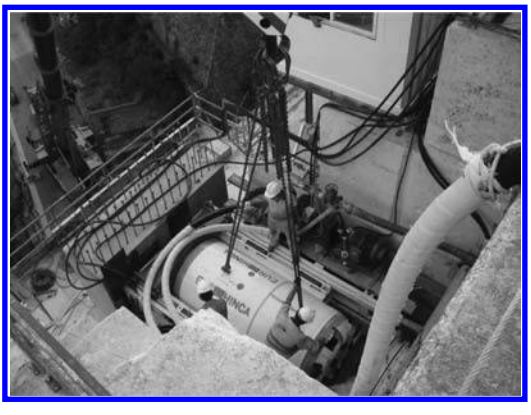

Figura 8.

El decantador se situó en una explanada existente en el estribo derecho de la presa con fácil acceso.

La caseta en la que se alojaban los equipos de control se coloco en una estructura creada aprovechando los escalones del paramento de aguas abajo de la presa, con visión directa de la zona de trabajo y anexa a la misma.

Por último se coloco el bastidor con el aro de empuje apoyando su parte de atrás solidaria al muro de empuje y embebiendo la pieza de ataque estanca.

Realizadas todas las conexiones eléctricas e hidráulicas todo estaba preparado para el inicio de la perforación.

La maquina comenzó a perforar el hormigón el día 10 de junio,

El funcionamiento del proceso de perforación es el siguiente.

La perforadora consiste básicamente en un cilindro metálico de 3.000 mm de longitud dividido en dos partes, una fija donde se alojan todas las conexiones, equipos hidráulicos, etc. que logran que la otra parte de la maquina, la cabeza rotativa perforadora, gire y se refrigere. Esta cabeza perforadora está dotada de una serie de "muelas" de widia adiamantada que al rozar con el hormigón lo van desmenuzando. A esta cabeza, a través de una tubería que circula por la parte interior de los tubos, llega agua impulsada por las bombas situadas en el cubeto de decantación. Este agua cumple dos misiones, por un lado arrastrar el detritus de la perforación desde la cabeza hasta el citado cubeto y por otro refrigerar la citada pieza móvil. Este sistema de entrada y salida de agua de la cabeza perforadora es controlado desde el puesto de mando.

La perforadora (ver Figura n° 7) necesita, además, un elemento que la empuje a medida que se va adentrando en la presa.

Este empuje lo realiza el anillo de empuje impulsado a su vez por dos cilindros soportados en el bastidor situado aguas abajo y apoyado en el muro de empuje.

Introducida hasta sus 2/3 partes la perforadora, se debe proceder a la colocación entre el anillo de empuje y la perforadora el tapón de estanqueidad. (Ver Figura n° 8)

Esta pieza consiste en un cilindro metálico de 1.500 mm de longitud, con una pared transversal en la que se aloja una escotilla con cierre hermético.

Durante los trabajos de perforación esta escotilla está abierta y por ahí atraviesan todas la tuberías de conexión a la perforadora y una vez la perforación llegue al lado del embalse y se hayan retirado las conexión es cerrada herméticamente.

Esta pieza, es la que una vez retirada la perforadora nos garantiza la estanquidad en el interior de la tubería. (ver Figura n° 9)

Esta pieza está unida aguas arriba con la perforadora mediante unión machihembrada con junta goma.

Colocada esta pieza y realizadas las conexiones se continua la perforación.

Introducida hasta sus 2/3 partes la pieza tapón, se procederá a la colocación del primer tubo de hormigón armado con camisa de chapa. (Ver Figura n° 10)

Este tubo esta unido tanto a la pieza tapón como al tubo siguiente y entre los sucesivos mediante unión machihembrada con junta goma.

Figura 9.

Figura 10.

Para controlar el avance de la cabeza perforadora, corrigiendo posibles desvíos se coloca un emisor laser en la perforadora y un receptor en un punto del muro de empuje. Este equipo esta monitorizado en el puesto de control y desde ahí, gracias a la pequeña capacidad de giro que tiene la cabeza de la perforadora, se fueron corrigiendo pequeñas desviaciones que iban surgiendo. Estas desviaciones en la línea de perforación, en principio se debían a la diferencia en la calidad del hormigón que la perforadora se iba encontrando en su avance. (ver Figura n° 11)

El ritmo de perforación fue el reflejado en la siguiente tabla:

LONGITUD DE PERFORACIÓN (metros)	DÍA
1,33	1
4,30	2
8,40	3
13,10	4
14,85	5
17,20	6

2.5 *Fase nº 5: Momento de "Cale"*

Dentro de toda esta descripción de la actuación llevada a cabo, cabe mención aparte el momento en el que la cabeza perforadora se encontrara con el agua del embalse, ya que en ese instante la presión que ejerce la columna de agua se traslada a la cabeza perforadora, a toda la superficie del cobrecorte y a la doble junta de estanqueidad de la pieza de ataque con el riesgo de fugas que esto conlleva.

Por los datos tomados con topografía se llegó a la conclusión de que la longitud de perforación era de 17,20 ml.

Figura 11.

Cuando por los datos que nos transmitía el equipo de medición laser la longitud de perforación era de 17,00 ml. procedimos a iniciar los preparativos para que el contacto con el agua del pantano fuera lo más suave y minimizar los posibles problemas.

Para ello se realizaron las siguientes acciones:

Se redujo la velocidad de giro de la cabeza perforadora para que la salida al agua fuera lo menos destructiva posible.

Por otro lado, entendiendo que la presión que se nos iba a transmitir era de 2,7 atm correspondiente a los 27 m.c.a. que teníamos aguas arriba utilizando el equipo de bombeo de aporta agua de refrigeración a la cabeza y extrae el agua con detritus de la misma, se elevo la presión existente entre la cabeza y el hormigón que nos quedaba sin demoler hasta las 2,3 atm lo que nos reduciría el golpe de ariete en el momento del "cale" y así reducir fracturas en hormigón o problemas en las juntas de estanqueidad aguas abajo como sucedió en actuaciones similares y que produjo problemáticas fugas.

Esta operación implicó un grave riesgo porque por un lado, implicaba reducir en gran medida la extracción de detritus que nos podía producir un taponamiento y bloqueo de la cabeza perforadora y por otro lado podría producirnos la rotura de las construcciones por la alta presión alcanzada.

En este momento un equipo de buzos se sumergió a la zona en que preveíamos iba a salir la perforadora para observar cualquier anomalía.

Alcanzada la presión en cabeza que deseábamos, y a una velocidad de giro 5 veces más lenta de la que veníamos llevando en el ritmo habitual, se comenzó el avance para la perforación de los 20 cm restantes.

En este momento un equipo de buzos se sumergió a la zona en que preveíamos iba a aparecer la perforadora para observar cualquier anomalía.

La perforadora alcanzaba lentamente a un ritmo 5 cm por hora. Todo discurría sin problemas y alrededor de la 3 hora de avance la presión del agua en cabeza subió hasta las 2,7 atm, lo cual era señal evidente de que el agua del pantano ya entraba en nuestra perforación.

Ya habíamos "calado" y todo había sucedido sin incidencias. No aparecía agua por las juntas de estanqueidad del sobrecorte, no aparecía agua por interior de las tuberías y solamente se incremento ligeramente el caudal en pequeñas fugas que ya existían en la zona de perforación. Todo había sucedido con éxito.

En este momento paramos el avance de la máquina para observar si surgía cualquier anomalía.

Se espero durante 2 horas y al no suceder ninguna incidencia se continuo la hinca del penúltimo tubo mediante el empuje del bastidor, ya con la máquina perforadora parada.

Hincado este tubo, se nos presentaba uno de los momentos más delicados de toda la operación, que era la colocación entre el anillo de empuje y el tubo hincado.

Para realizar esta operación el anillo de empuje debía liberarse de todo el conjunto de tubería hincada que en ese momento ya recibía el empuje del agua del embalse.

Para soportar este empuje y que todo el conjunto no nos retrocediera se colocaron 8 cartelas que unían mediante electrosoldadura el extremo metálico hembra del tubo nº 5 con la pieza de ataque estanca que a su vez estaba unida al chasis embebido en el muro de hormigón armado.

Finalizado el proceso de soldadura, se procedió a retroceder el anillo de empuje 1 cm. para observar la reacción del conjunto hincado. Se espero durante 2 horas la evolución no registrándose el sensor de movimiento ninguna oscilación. (Ver Figura n° 12)

Rápidamente se procedió a la total retirada del anillo de empuje, a la colocación del tubo nº 6 (y último), al ensamblaje del anillo en este tubo y a la retirada de las cartelas de sujeción.

La operación se realizo con éxito y seguidamente se debía proceder al empuje del conjunto para la extracción de la cabeza perforadora.

2.6 *Fase n°6: Extracción de la perforadora*

Este avance debía ser lento y sincronizado entre el empuje del bastidor y la salida de la perforadora por el agujero "de aguas arriba".

La perforadora estaba dotada en su chasis de dos puntos de amarre que facilitaban el enganche del sistema de elevación de la misma a superficie.

Esta elevación se realizo con una grúa de gran tonelaje guiada por las instrucciones de los equipos subacuáticos.

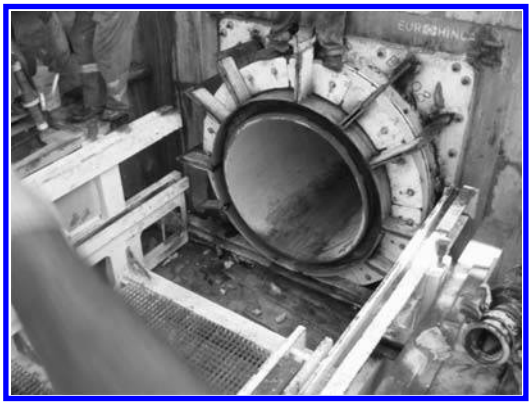

Figura 12.

La secuencia de avance fue la siguiente.

La perforadora se empujo hasta que el primer punto de amarre (0,80 ml de recorrido) asomo aguas arriba del paramento de aguas arriba de la presa. En ese momento, los buzos procedieron al enganche de la perforadora a las cadenas de la grúa.

A partir de ese momento la pieza iba a avanzar impulsada por el bastidor y soportada en su parte delantera por la grúa. Cualquier movimiento brusco podía implicar la separación de las uniones machihembrados de los tubos y la consiguiente entrada de agua al interior de la conducción.

Lentamente se avanzo hasta que el segundo punto de amarre apareció en el agua (2,20 ml de recorrido), en este punto se volvió a detener el avance y el equipo de buzos engancharon las cadenas en el segundo punto de amarre.

En este momento la perforadora estaba ya totalmente soportada por la grúa, pero debíamos sincronizar los movimientos hasta que toda la perforadora estuviera fuera de la perforación.

El avance prosiguió hasta que toda la perforadora ya asomaba fuera de la perforación, sin embargo continuamos el avance hasta que la pieza tapón asomo aproximadamente 0,10 ml.

En este momento la hinca de la tubería había finalizado y solo restaba separar la perforadora de la pieza tapón.

Para ello la citada pieza tenia instalados unos cilindros en su parte interior, que tras la conexión de los manguitos desde superficie, impulsaron la perforadora y la separaron sin ningún tipo de incidencias de la pieza de estanqueidad.

Finalmente la grúa elevo a superficie la perforadora, para alegría de todo el equipo humano de la obra.

2.7 *Fase nº 7: Inyección del sobre corte*

Finalizado todo el proceso de perforación, se procedió a rellenar el espacio entre la tubería hincada y el perímetro de perforación. Esta corona circular tenia un espesor medio de 17,5 mm. Para el relleno de este sobrecorte se utilizaron resinas acuoreactivas.

Estas resinas fueron inyectadas con un pequeño equipo de inyección a través de unos pequeños inyectores que se colocaron en los alojamientos dispuestos radialmente para tal fin en los tubos de hormigón con camisa de chapa (3 inyectores por tubo). Esta inyección se inicio en el primer tubo aguas abajo y secuencialmente se prosiguió la inyección hacia aguas arriba, para así expulsar el agua que se alojaba en el sobre corte hacia el pantano. Se utilizaron dos tipos de resinas:

APOGEL PU-20: Esta primera resina reaccionaba con el agua transformándose en una espuma de poliuretano muy porosa que ocupaba mucho espacio expulsando el agua, pero dejando grandes huecos en su interior.

APOGEL FLEX: Esta segunda resina fue utilizada para rellenar los huecos resultantes de la primera inyección. Esta resina presentaba un alto poder de adherencia y muy baja viscosidad lo que nos garantizaba que todos los espacios y huecos inter espuma iba a quedar relleno y además se iba a aportar algo de adherencia al conjunto.

Cuando en el agua del embalse y en la vertical de la perforación apareció espuma de poliuretano se dio por concluida la primera fase de inyecciones.

2.8 *Fase nº 8: Anclaje de tubería*

Finalizada la ejecución del relleno se iniciaron los trabajos de anclaje da la tubería, para así retirar el bastidor que desde el momento del "cale" era el que soportaba el empuje que los 27 m.c.a. ejercían en la pieza tapón.

De los cálculos realizados se desprendió que para compensar el citado empuje era necesaria la colocación de 17 anclajes Ø16 L = 0,5 + 0,15 ml.

Para la colocación de estos anclajes se procedió a la retirada de los inyectores que estaban alojados en las paredes de los tubos y por ese agujero se procedió ala perforación en la pared de hormigón de la presa. Los citados anclajes que finalmente, por distribución fueron 18 Ud. se unieron a la perforación en la pared mediante resina epoxi de alta adherencia.

2.9 *Fase nº 9: Retirada de bastidor de empuje y corte de exceso de tubería*

Pasados unos días para que la resina endureciera, se debía proceder a la retirada del bastidor.

Este momento lo entendíamos como de gran riesgo por lo cual se tomaron especiales precauciones.

Antes de cualquier movimiento. Se procedió a la colocación de un sensor de movimiento en la parte de la tubería que sobresalía aguas abajo.

Seguidamente de procedió lentamente a la descarga de presión en los bombines lo que supuso el retroceso del anillo de empuje 1 cm. Realizado este movimiento se espero durante 4 horas para observar si había algún tipo de movimiento. No observándose ningún movimiento se procedió a la retirada del anillo de empuje hasta su tope de desplazamiento.

Con la zona de trabajo despejada, se debía cortar parte del último tubo hincado que sobresalía aguas abajo, hasta dejar solamente 0,30 ml donde posteriormente alojaremos el casquillo de atornillado de la válvula Bureau.

Para ello y dado el poco espacio del que disponíamos se opto por la opción del corte mediante

hilo de diamante, que necesito de un complicado conjunto de poleas para adaptarlo al punto de trabajo.

Retirado el exceso de tubería se procedió al desmontaje y posterior retirada del todo el equipo de perforación.

2.10 *Fase nº 10: Colocación de casquillo de atornillado de válvula Bureau*

El elemento de cierre de la conducción hincada es una válvula bureau de diámetro 1000 mm. Esta válvula debe unirse a la conducción mediante una brida en forma de corona circular con 28 orificios para colocación de otros tantos tornillos de M-16. Por lo que se nos presentaba el problema de unir esta corona con nuestro tubo de hormigón. (Ver figura n° 13)

La solución que se le dio al problema fue un tubo de acero de 0,50 ml de longitud que en uno de sus extremos tenia soldada una corona circular idéntica a la que porta la válvula y por el otro, este tubo se unió mediante electrosoldadura con la pieza de ataque estanca que en su momento se había colocado solidaria al hormigón de la presa. Para dotar a esta unión de la necesaria estanqueidad de procedió a la unión mediante electrosoldadura del tubo metálico de unión con la chapa del encamisado del tubo de hormigón hincado.

2.11 *Fase nº 11: Colocación de válvula Bureau y continuación de conducción*

Seguidamente se procedió a la colocación de la válvula Bureau. Esta operación requirió de una grúa de gran tonelaje.

A continuación de la citada válvula se prosiguió por un lado con la colocación de la tubería de acero inoxidable Ø1000 mm hasta el pie de presa y por otro con el recubrimiento mediante hormigón de la válvula Bureau. (Ver figura n° 14)

Figura 13.

Figura 14.

2.12 *Fase nº 12: Retirada de la Pieza-Tapón*

Como se ha citado anteriormente, la pieza tapón (de 1.500 mm de longitud) en las operaciones de extracción de la cabeza perforadora se extrajo hacia aguas arriba del paramento de la presa 150 mm.

Una vez la que la válvula Bureau nos garantizaba la estanquidad de la conducción, debíamos proceder a la extracción de la pieza-tapón para así poner en servicio la conducción. (Ver figura n° 15)

Para esta operación debíamos contar otra vez con la colaboración del equipo de buceo.

La primera acción era poner en carga la tubería, para eso se procedió por un lado a la apertura de un pequeño purgador que existe en la válvula Bureau para la evacuación del aire de la tubería y por otro a la apertura de una válvula de entrada de agua dispuesta para este fin en la pieza tapón.

Conseguido el llenado de la conducción, se procedió a la conexión de un grupo oleo hidráulico situado en superficie a unos cilindros hidráulicos que se habían colocado previamente al proceso de hincado en el primer tubo de la hinca. El empuje de estos cilindros consiguió la extracción de la pieza tapón en un recorrido de 750 mm, con lo cual finalizada la operación con los cilindros (con los que se debió ejercer una presión muy por encima de lo esperable) teníamos la pieza 1000 mm fuera del paramento de la presa.

Pero todavía quedaba la pieza 500 mm alojada en el conducto.

Figura 15.

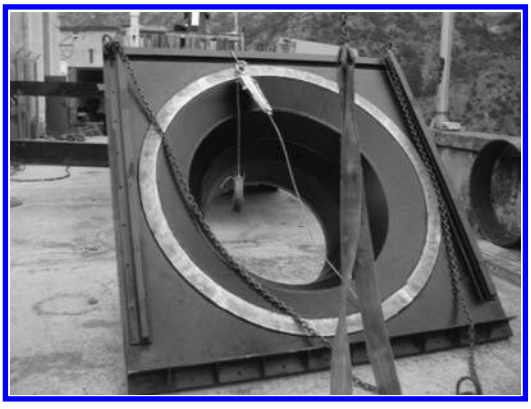

Figura 16.

En principio esto no era ningún problema ya que la holgura que nos daba el sobrecorte calculábamos era suficiente para la extracción de la pieza tapón con la ayuda de una grúa situada en superficie y guiada por las indicaciones del equipo de buceadores situados en el punto de salida de la perforación.

Iniciada la operación los buceadores observaban que no se producía ningún tipo de movimiento en la pieza tapón respondiendo a los movimientos que realizábamos con la grúa desde superficie.

De una minuciosa observación los buzos observaron que el espacio del sobrecorte estaba colmatado con las resinas endurecidas inyectadas previamente. Esto hacía imposible cualquier tipo de movimiento de la pieza tapón.

Ante esta situación, se analizó el problema y se decidió proceder a la demolición mediante equipos especiales subacuáticos del perímetro de la pieza tapón.

2.13 *Fase nº 13: Colocación de pieza embocadura y demolición previa.*

Para facilitar la entrada de agua a la conducción y para que esta funcione a máximo rendimiento se diseño una pieza de embocadura metálica. (Ver figura n° 16)

Para alojar esta pieza en la salida de la conducción era necesario proceder a la demolición de un anillo circular de 0,50 ml de anchura y 0,50 ml de profundidad.

Esta demolición en principio estaba previsto fuera ejecutaba una vez extraída la pieza tapón, pero debido al problema anteriormente citado se ejecuto previamente.

Finalizada la demolición, la extracción de la pieza tapón se realizo sin ningún tipo de problemas.

Seguidamente se procedió a la colocación de la embocadura. Esta embocadura se debía unir al conjunto de tubos hincados.

Para ello la citada embocadura llevaba insertada una arandela con 20 agujeros pasantes equidistantes dispuestos radialmente.

Previamente y durante la fabricación de los tubos de hormigón armado con camisa de chapa, el tubo que iba a ser el primero en hincar, y que obviamente seria al que debíamos unir la embocadura se fabrico con su parte “macho” en acero inoxidable y con una ranura radial con 20 agujeros roscados M-10 equidistantes dispuestos radialmente, donde debía alojarse la arandela insertada en la embocadura y atornillar en los citados agujeros los correspondientes tornillos.

Esta operación, dado el volumen y el peso de la citada embocadura, y la poca tolerancia de la que disponíamos se preveía delicada pero sin embargo se realizo con rapidez y éxito.

Encajada la embocadura en la ranura de extremo metálico del tubo se procedió al anclaje de la citada embocadura mediante 40 anclajes M-16 y L = 120 mm.

2.14 *Fase nº 14: Inyección del espacio entre embocadura y perímetro de demolición*

Para finalizar la actuación se procedió a rellenar el espacio entre la embocadura y el perímetro de perforación. Para el relleno de este espacio se utilizaron las mismas resinas acuoreactivas que en la inyección del sobrecorte de la perforación

Cuando la resina inyectada por el inyector inferior apareció en forma de espuma de poliuretano por los superiores de dio por concluidas las inyecciones.

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Diversas actuaciones realizadas en las presas que regulan el río Cinca

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RESUMEN: El artículo describe las obras proyectadas en la presa de Mediano para su adecuación al RTSPE, que consisten en el acondicionamiento y remodelación del canal de descarga del aliviadero y el trampolín, la remodelación del cuenco amortiguador, la estabilización de laderas y otras obras complementarias. También se recogen las obras de emergencias en la presa de El Grado para garantizar la seguridad de la misma mediante la rehabilitación de los desagües de fondo de la presa, reparando las antiguas compuertas de regulación, dejándolas como compuertas de seguridad, y ampliando los cuatro conductos instalando unas nuevas compuertas de regulación.

ABSTRACT: The article describes the planned work on the Mediano dam for adaptation to RTSPE, involving the refurbishment and remodeling of the spillway discharge channel and the skijump, the remodelling of the water cushion, stabilization of slopes and other complementary works. Emergency works at El Grado dam are also collected in the project in order to ensure the dam safety by restoring the bottom outlet, repairing the old regulation gates, leaving them as security gates, and enlarging the four channels installing new regulation gates.

1 INTRODUCCIÓN

El embalse de El Grado, junto con el embalse de Mediano, regulan el río Cinca, afluente del Segre que a su vez es afluente del Ebro, abasteciendo el Sistema de Riegos del Alto Aragón a través del Canal del Cinca, que se une al Canal de Monegros en Tardienta.

Dada la importancia de ambas presas, es necesario mantenerlas en perfecto estado de conservación y de funcionalidad.

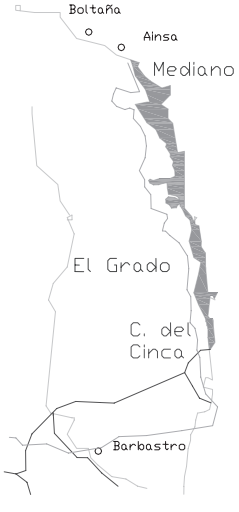

Figura 1. Plano de situación de las presas en el río Cinca.

2 DESCRIPCIÓN DE LAS PRESAS

La cuenca del río Cinca se encuentra en la provincia de Huesca, en su parte nororiental, teniendo su inicio en los Pirineos. Los principales elementos de regulación del río son las presas de Mediano y El Grado.

La finalidad principal de la presa de Mediano es el riego a través del canal del Cinca que a su vez suministra agua al canal de Monegros. La regulación de agua para riegos se hace de forma conjunta con el embalse de El Grado, desde donde arranca el canal del Cinca. Su cola de embalse llega al pie de la presa de Mediano.

La finalidad del embalse de El Grado es aumentar el volumen de agua del río Cinca regulada para riegos a través del canal del Cinca, sirviendo además de contraembalse a la presa de Mediano.

2.1 *Presa de Mediano*

La presa de Mediano se sitúa sobre el río Cinca, afluente del Segre que a su vez lo es del Ebro por su margen izquierda. Está situada en el término municipal de La Fueva en la provincia de Huesca.

Tiene una capacidad de 436,35 Hm^3 con NMN a la cota 528 msnm. La tipología de la presa es de gravedad de hormigón, con un talud del 0,06 en el paramento de aguas arriba y de 0,793 en el de aguas abajo. La altura sobre cimientos es de 92 m mientras que sobre el cauce es de 74 m.

La principal singularidad de la presa es que está formada por un recrecimiento en el paramento de aguas arriba de una presa más antigua. Se tiene entonces un cuerpo de presa formado por el hormigón de la presa antigua (finalizada en 1960) más el correspondiente al recrecimiento (finalizado en 1973). La planta de la presa está compuesta de tres alineaciones rectas unidas por dos curvas circulares.

Por lo expuesto anteriormente la presa tiene dos coronaciones. La coronación de la presa correspondiente al recrecimiento se sitúa a la cota 529,50 msnm. Tiene una longitud de 500 m con una anchura total de 5 m, con calzada de firme de aglomerado asfáltico y aceras a ambos lados de 0,75 m. En la presa primitiva la coronación de la presa estaba situada a la cota de 521,30 msnm.

La cerrada sobre la que está construida la presa es marcadamente asimétrica, con un estribo derecho de gran longitud (215,50 m) donde se sitúa el aliviadero. Los desagües se sitúan en el estribo izquierdo (113,94 m), donde la presa tiene mayor altura. La toma de agua para la central hidroeléctrica está situada fuera del cuerpo de presa y muy próxima a su estribo izquierdo.

El aliviadero está situado en la alineación derecha de la presa sobre la coronación de ésta. Está constituido por cuatro vanos de 11,70 m de ancho cada uno de ellos regulados por compuertas Taintor. El perfil del vertedero es de tipo Creager y el canal de descarga, común a todos los vanos, es de conducción libre sobre el cuerpo de presa y la roca de la ladera derecha. Este vierte en el cauce del río en un cuenco natural mediante un trampolín con dientes.

El ancho del canal de descarga es de 57,80 metros y su longitud, medida a partir del paramento de aguas abajo de la presa es de 132,50 m. La altura de los muros cajeros es de 4,50 m, a excepción del arranque de éstos en donde tienen una altura variable debido a un recrecimiento realizado. Se desarrolla en el macizo rocoso en que se apoya el estribo derecho de la presa y acaba con unos dientes disipadores de energía que forman un trampolín, desembocando en un cuenco natural situado por debajo del pie de la presa. En dicho cuenco desembocan igualmente los desagües de fondo y medio fondo.

Figura 2. Vista general de las presa de Mediano y sus órganos de desagüe.

El desagüe de medio fondo tiene la cota de eje de compuertas a 463,50 msnm. Está compuesto por dos conductos gemelos, dotado cada uno de ellos por dos válvulas compuerta tipo Bureau de 1,00 × 1,00 metros (seguridad y regulación) con una capacidad máxima de desagüe de 25 m^3/s por conducto.

En la misma cámara que el desagüe de medio fondo, pero a diferente cota (la cota del eje de compuertas es 453,85 msnm), se encuentra el desagüe de fondo de la presa, formado por tres conductos gemelos, dotados cada uno de ellos por dos válvulas compuerta tipo Bureau 1,00 × 1,50 metros (seguridad y regulación).

La salida al río se produce en la margen izquierda, aguas arriba de la salida de la central hidroeléctrica y a un nivel por encima de la salida del desagüe de fondo.

2.2 *Presa de El Grado*

La Presa de El Grado está construida sobre el río Cinca, afluente del Segre, que a su vez lo es del Ebro por la margen izquierda. Está situada en el término municipal de El Grado, del cual recibe el nombre, provincia de Huesca. Fue terminada de construir en 1969 y tiene una capacidad de 400,00 Hm^3.

La tipología de la presa es de gravedad de hormigón, de planta formada por cuatro alineaciones rectas unidas por tres curvas circulares y su altura máxima sobre cimientos es 130,00 metros. La cota de nivel máximo normal (NMN) es la 450,00 msnm. La altura sobre cimientos es de 130 m mientras que sobre el cauce es de 88 m.

El aliviadero está situado en el cuerpo de la presa en la coronación de la presa, ocupando la alineación central en planta en toda su longitud. Está constituido por tres vanos de 20 m de ancho cada uno de ellos regulados por compuertas Taintor. El perfil del vertedero es de tipo Creager y el canal de descarga, común a todos los vanos,

Figura 3. Vista general de la presa de El Grado.

es de conducción libre sobre el cuerpo de presa, vertiendo en un cuenco amortiguador.

La toma del canal del Cinca está situada en el estribo derecho de la presa, junto a la toma de la central hidroeléctrica de El Grado I. Tiene 3 conductos. Los dos laterales de 2,80 m de diámetro están regulados cada uno de ellos por una compuerta vagón de 2,90 × 2,90 m que actúa a modo de ataguía y una válvula Larner-Johnson como cierre de regulación colocadas en serie. El conducto central de 3,70 m de diámetro dispone de una compuerta vagón de 3,90 × 3,90 m de ataguía y una compuerta Bureau de regulación de 1,60 × 2,50 m.

En la presa de El Grado hay dos desagües de fondo instalados en los túneles de desvío del río en la ladera izquierda de la presa. Cada uno de ellos consta de 2 conductos, cada uno de los cuales, después de terminar las obras objeto de este artículo, está regulado por tres compuertas tipo Bureau colocadas en serie. La de aguas abajo de regulación, una de seguridad aguas arriba de la anterior y una ataguía aguas arriba de las dos anteriores. Las tres tienen las mismas dimensiones de 1,25 × 1,80 m. El caudal máximo para cada desagüe con las compuertas abiertas en su totalidad con el nivel del embalse en el NMN es de 85,26 m^3/s.

3 OBJETO DE LAS ACTUACIONES

A continuación se detalla el objeto de ambas actuaciones.

3.1 *Presa de Mediano*

En la presa de Mediano se han encontrado una serie de deficiencias para su adecuación al Reglamento Técnico de Seguridad de Presas y Embalses (RTSPE).

Las principales deficiencias existentes en la presa de Mediano son el estado del hormigón del canal de descarga del aliviadero y del trampolín que se encuentra muy deteriorado. Además, existe la necesidad de remodelar el cuenco amortiguador. También es necesario realizar una serie de mejoras a las salidas de los desagües de medio fondo y en el hormigón del paramento aguas abajo.

Todas estas circunstancias aconsejan renovar el canal de descarga y reconstruir el cuenco amortiguador del aliviadero, para lograr un adecuado funcionamiento de las instalaciones.

3.2 *Presa de El Grado*

Durante el año 1999 se observó que los desagües de fondo habían dejado de ser operativos, produciéndose un fenómeno de eutrofización en las aguas de cotas bajas, una rápida sedimentación de sólidos por falta de circulación con el consecuente aterramiento por la acumulación de lodos, pudiendo acarrear una anulación total de la operatividad de los propios desagües en un futuro. Por lo que se planteó la reparación para evitar daños a la presa y al cauce del río.

En octubre de 2000, al plantear la sustitución de las compuertas de regulación, se puso de manifiesto la imposibilidad de la reparación de las compuertas de cierre, a las que habría que confiar el cierre del desagüe. Este hecho provocó las obras de emergencia de rehabilitación total de los desagües de fondo e incorporación al río Cinca y de implantación de nuevas compuertas de regulación en los desagües de fondo de la presa de El Grado.

La actuación sobre los desagües de fondo consistió en la reparación de las antiguas compuertas de seguridad para tener un funcionamiento como ataguías. Además se repararon los cierres de regulación para dejarlos como compuertas de seguridad, y por último se instalaron unas nuevas compuertas de regulación aguas abajo reformando la salida de estas últimas mediante la colocación de un nuevo blindaje en el inicio de los túneles, garantizando así la seguridad de la presa.

Para el remate final de la obra, fueron necesarias obras accesorias como la demolición y ampliación de la cámara de compuertas, la instalación de nuevos grupos oleohidráulicos, aireaciones, blindajes y la reposición de instalaciones eléctricas.

4 DESCRIPCIÓN DE LOS TRABAJOS A REALIZAR

A continuación se detallan los trabajos realizados en las actuaciones relatadas.

4.1 *Presa de Mediano*

Las principales actuaciones previstas en el proyecto de adecuación al RTSPE de la presa de Mediano son las siguientes:

- Acondicionamiento y remodelación del canal de descarga del aliviadero y el trampolín
- Remodelación del cuenco amortiguador
- Estabilización de laderas
- Obras complementarias

4.1.1 *Acondicionamiento y remodelación del canal de descarga del aliviadero y el trampolín*

En el canal de descarga se ha proyectado el saneado del hormigón del canal de descarga del aliviadero, picando inicialmente todo el canal con una profundidad mínima de 25 a 30 cm, y después procediendo a su hormigonado, manteniendo juntas de dilatación tanto longitudinales como transversales.

El saneado se realizará sobre las partes huecas o mal adheridas del hormigón, repicando el hormigón en las zonas que presenten manchas de óxido, grasas y aceites, así como en aquellas zonas en las que el hormigón esté hueco o mal adherido, hasta encontrar un soporte firme y resistente.

Seguidamente se procederá a la limpieza de la superficie de polvo o lechadas superficiales mediante algún sistema de chorro de arena, agua o mixto.

Se acometerá la limpieza de las armaduras vistas eliminando el óxido existente mediante chorro de arena y cepillo de púas metálicas. Una vez limpias las armaduras se pasivarán con un revestimiento de adherencia y protección de armaduras a base de productos de cemento y mejorados con resina sintética y humo de sílice.

Previo al vertido de la nueva capa de hormigón, se aplicará sobre al hormigón viejo un adhesivo a base de resinas epoxi que proporciona una perfecta unión con el hormigón nuevo.

Para asegurar la conexión del hormigón con la nueva capa se procederá a la colocación de conectores anclados al hormigón. Estos conectores serán barras corrugadas de 8 mm de diámetro con forma en L. Sus dimensiones serán de 120 mm de altura y patilla de 60 mm. Se procederá al anclaje en el hormigón ejecutando un taladro de 12 mm de diámetro y profundidad de 90 mm, que se rellenará con resina epoxi de anclaje antes de introducir el conector.

Se ha proyectado la elevación del trampolín y los 8 hitos lanzadores con las dimensiones y características diseñadas según el estudio realizado con el modelo reducido. Se dotará de pendiente a la zona superior de los muros cajeros mediante mortero resistente al agua.

A lo largo de la junta de unión de la solera del canal con los muros cajeros se dispondrá una lámina de unión para evitar posibles erosiones, dado que esta zona es más sensible a ello.

Los trabajos consistirán en la remodelación de todo el canal de descarga del Aliviadero, incluyendo sus muros. Así mismo se construirán ocho dientes (eliminado los 6 existentes actualmente, uno de ellos desprendido en el cuenco amortiguador).

Los detalles de inclinación en la parte superior de los cajeros y de la unión de los muros del cajero con la solera del canal de descarga deberán detallarse al ser un diseño nuevo.

En todo momento se referirá a la futura modelización del funcionamiento del canal de descarga, trampolín y cuenco amortiguador por parte del CEDEX para variaciones en el diseño del mismo.

4.1.2 *Remodelación del cuenco amortiguador*

Se han proyectado trabajos de limpieza del mismo (mota de tierra existente actualmente), y aprovechando los materiales retirados y la aportación de los necesarios, se construirá aguas abajo una contraataguía unos metros antes de la salida de la Central, con el fin de evitar el vertido de posibles materiales a la salida de la Central.

Dicha contraataguía será de materiales sueltos con pantalla hormigonada aguas abajo. Sus accesos serán suficientemente amplios para la inspección por parte del personal de la presa, y la cota será la de proyecto (452,50 msnm, misma cota que la salida del desagüe de fondo).

En principio no es necesaria la limpieza total del cuenco, ya que no se requiere una cota exacta de la solera del cuenco.

4.1.3 *Estabilización de laderas*

Tanto en la ladera izquierda como en la derecha se ha proyectado el desbroce y saneado de la ladera, y la estabilización mediante malla metálica, construyendo un muro de contención, situado en la base la de ladera, como protección al cuenco con losas de gran tamaño.

4.1.4 *Obras complementarias*

Será necesario la reconducción del desagüe de medio fondo (cota 463,00 msnm), hacia el centro del cuenco para amortiguar la energía, mediante la construcción de unos muros direccionados al centro del cuenco. También se realizará el saneamiento de las zonas necesarias del hormigón del paramento aguas abajo.

Se ha proyectado la reconducción del canal de salida de los desagües de medio fondo mediante la construcción de dos muros cajeros de encauzamiento del desagüe de medio fondo, aislando su descarga de la del desagüe de fondo, evitando así una mayor erosión en el cuenco amortiguador.

Figura 4. Acondicionamiento y remodelación del canal de descarga del aliviadero y el trampolín.

Figura 5. Remodelación del cuenco amortiguador y construcción de contraataguía.

Figura 6. Estabilización de laderas y reconducción del canal de salida de los desagües de medio fondo.

Se procederá al saneo del hormigón del paramento de aguas abajo de la presa mediante morteros de alta resistencia.

Se procederá a las actuaciones y mejoras necesarias en las instalaciones de la Casa de Administración y las instalaciones auxiliares.

4.2 *Presa de El Grado*

Previo a la ejecución de las obras de emergencia, se procedió al cierre de las compuertas de seguridad y la obturación de los conductos mediante el retacado de cada uno de ellos utilizando un robot subacuático. El resto de las fugas existentes se condujeron a través de tubos hasta la salida de los cierres.

Las actuaciones en las dos obras de emergencia ejecutadas fueron las que se describen a continuación.

4.2.1 *Rehabilitación total de los desagües de fondo e incorporación al río Cinca*

La rehabilitación de los desagües de fondo consistió, principalmente, en actuaciones en la cámara de válvulas y en los túneles. Se ejecutó el acondicionamiento de los accesos a los túneles para facilitar la obra.

Se procedió a la demolición de fábrica de hormigón en la cámara de válvulas en dos fases, una primera en la zona superior y la segunda en la zona inferior para poder alojar las nuevas compuertas de regulación. La demolición se realizó por medios mecánicos con la extracción de escombros a vertedero autorizado. Esta actuación requirió de sistemas de aspiración eléctricos, dado que se trabajaba en un espacio cerrado.

En la cámara de válvulas se revisó el sistema de accionamiento oleohidráulico de las compuertas de seguridad en los cuatro conductos. Además se incorporaron by-passes nuevos con válvula compuerta Ø150 para dotarlos doble cierre. En el sistema de aireación de las compuertas de seguridad se incorporaron válvulas compuerta y ventosas de Ø200.

En los cierres de regulación se desmontaron los by-passes, tuberías de aireación, servomotores, tableros y campanas para su revisión, procediendo posteriormente a la instalación de nuevas camisas, vástagos, campanas, tableros con sus guarniciones y elastómeros. El sistema de accionamiento se renovó totalmente mediante la instalación de dos nuevos grupos oleohidráulicos, uno por cada túnel, con la instalación de tuberías oleohidráulicas de conexión en acero inoxidable y un armario eléctrico con el aparellaje de protección, mando y control.

Se colocó un blindaje especial aguas abajo de las compuertas existentes para el posterior alojamiento de las compuertas Bureau, con los

refuerzos y bulones necesarios para soportar las presiones de trabajo, así como un blindaje especial en la solera del túnel a la salida de las nuevas compuertas de regulación para evitar el desgaste producido por el impacto del agua en las maniobras de las compuertas.

El sistema de aducción de aire para la futura compuerta de regulación se conectó al colector general.

En la zona de los túneles, se ejecutó un blindaje mediante acero en perfiles laminados y en palastro, laminado y soldado. También se ejecutaron unos orificios para inyección tras el revestimiento de acero.

Una vez acabadas las actuaciones anteriores y preparadas las superficies de unión con los hormigones existentes, mediante chorro de arena y agua a presión, se procedió al hormigonado con bombeo hidráulico en el interior de las galerías de servicio y cámara de válvulas.

En toda la obra se contó con un sistema de agotamiento en la cámara para disminuir la presencia de agua. Además se disponía de sistemas de ventilación y alumbrado compuestos por grupo electrógeno y las instalaciones necesarias para un correcto y seguro funcionamiento.

4.2.2 *Implantación de nuevas compuertas de regulación en los desagües de fondo de la presa de El Grado*

Se ejecutaron demoliciones en la zona inferior de la cámara de válvulas y galerías, ampliándola para permitir el alojamiento de las nuevas compuertas de regulación. Las superficies se prepararon mediante limpieza con chorro de arena y agua a presión para su posterior hormigonado.

Se realizó la instalación y montaje de cuatro nuevas válvulas compuertas tipo Bureau de regulación con sus correspondientes accionamientos mediante conducciones oleohidráulicas, grupo oleohidráulico, by-passes y tubería de aireación.

Las principales actuaciones fueron las nuevas compuertas de regulación con nuevos tableros, guarniciones y elastómeros, los cricks, camisas Ø400, vástagos Ø145, campanas y los perfiles para los cuerpos de las válvulas compuertas. También se procedió al pintado de los cilindros y cúpulas y la incorporación de dos grupos doble motor-bomba de 10 C.V. uno por túnel, para el accionamiento de los cierres de seguridad y regulación.

A continuación se detallan cada una de las fases en que se ejecutaron las obras.

FASE I: Retacado de fugas con robot subacuático, aguas arriba de las compuertas de seguridad, para disminución de fugas existentes en cada uno de los cuatro conductos.

FASE II: Desmontaje de la compuerta de regulación y sustitución de tablero, cúpula y cilindro oleohidráulico.

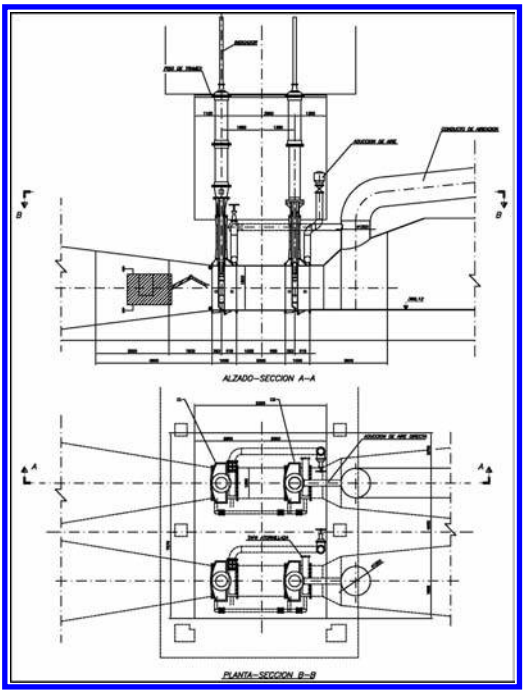

Figura 7. Fase I. Retacado de fugas con robot.

En esta fase se procedió al desmontaje de los elementos de la compuerta para su sustitución por otros de nueva generación, más fiables y funcionales.

Para cada una de las cuatro compuertas antiguas de regulación se desmontaron cricks, campanas y tableros para su sustitución por nuevos dado su nivel de degradación. También se procedió al desmontaje de la instalación oleohidráulica.

FASE III: Demolición en zona superior de cámara de válvulas.

Se ejecutaron demoliciones en la zona superior para conseguir el espacio necesario para la instalación de la conexión del sistema de aireación al conducto de aireación existente.

FASE IV: Montaje y ajuste de nuevos tableros, cúpulas y cilindros oleohidráulicos de las compuertas de regulación.

Tras el desmontaje de los elementos en las compuertas de regulación, se procedió al montaje y ajuste de los nuevos tableros, cúpulas y cilindros de camisa Ø400 y vástago Ø145, de nueva generación con guarniciones y elastómeros nuevos para asegurar un mejor cierre de las compuertas. Se mantuvo el sistema de aireación mediante tubería de aireación Ø200, válvulas compuerta y ventosa Ø200.

FASE V: Demolición de la zona inferior de la cámara de válvulas.

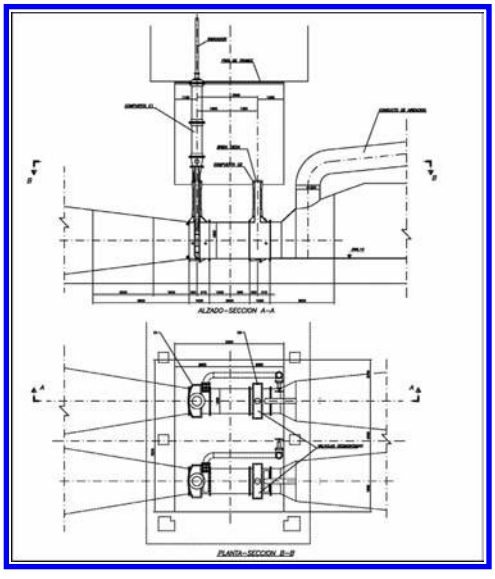

Figura 8. Fase II. Desmontaje compuerta regulación.

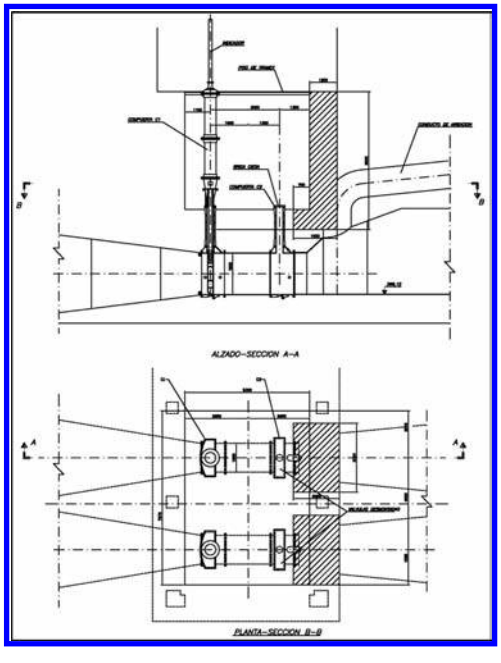

Figura 9. Fase III. Demolición en zona superior de cámara.

En esta fase se demolió la zona inferior de la cámara de válvulas para lograr el espacio suficiente para instalar las nuevas compuertas de regulación de los desagües. Además se ejecutaron unos conductos laterales de aireación desde la parte inferior a la salida del blindaje de las compuertas

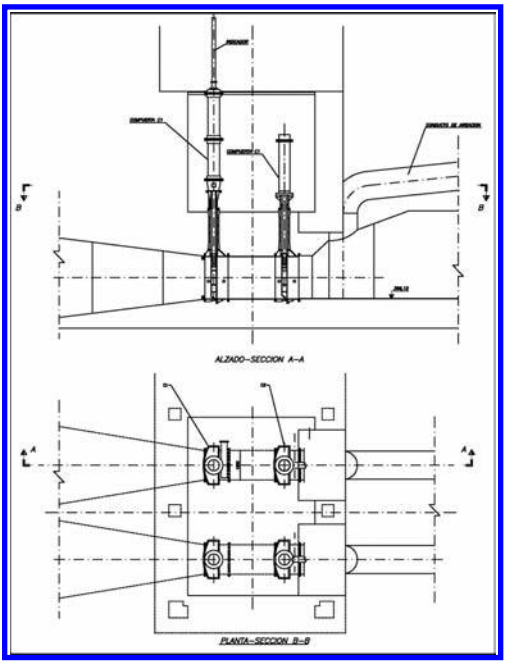

Figura 10. Fase IV. Montaje y ajuste de nuevo tablero, cúpula y cilindro oleohidráulico.

de regulación, en un escalón situado en su zona inferior, conectados al conducto de aireación existente.

FASE VI: Montaje de nuevas compuertas de regulación, aducciones y blindajes de salida.

Una vez creado el espacio para la instalación de las nuevas compuertas de regulación se procedió al montaje de las compuertas, mediante la colocación de un carrete entre compuertas de seguridad y regulación. Se montaron las tuberías de aireación de las compuertas de regulación conectadas a la conducción de aireación de hormigón existente y por último los blindajes a la salida de los cierres de regulación, dejando un escalón a la salida de estos para mejorar la aireación de los mismos.

FASE VII: Hormigonado de nuevas compuertas y blindajes.

Tras el montaje de las compuertas se procedió al hormigonado tanto en la zona superior, sistema de aireación, como en la zona inferior, entre compuertas y a la salida del blindaje del cierre de regulación. También se colocó un blindaje de protección en la solera de los túneles a la salida de los conductos para proteger del desgaste producido por el impacto del agua.

FASE VIII: Instalaciones accesorias, renovación de aireaciones y by-passes.

Por último se instalaron el resto de elementos necesarios para el correcto funcionamiento de las

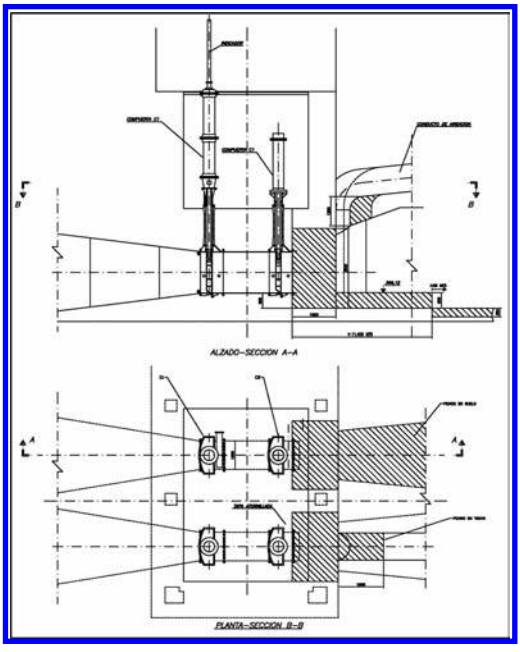

Figura 11. Fase V. Demolición en zona inferior de cámara.

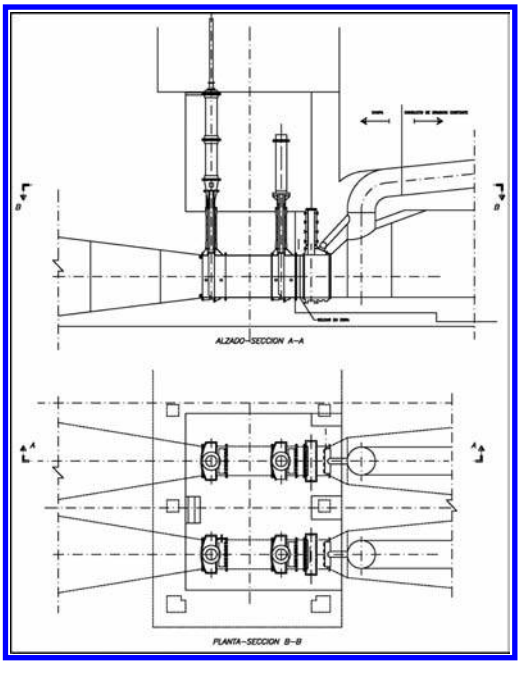

Figura 12. Fase VI. Montaje nuevas compuertas de regulación.

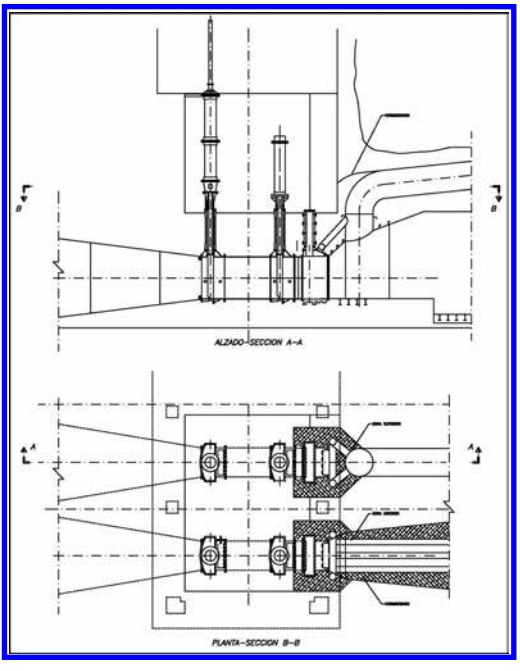

Figura 13. Fase VII. Hormigonado de nuevas compuertas y blindajes.

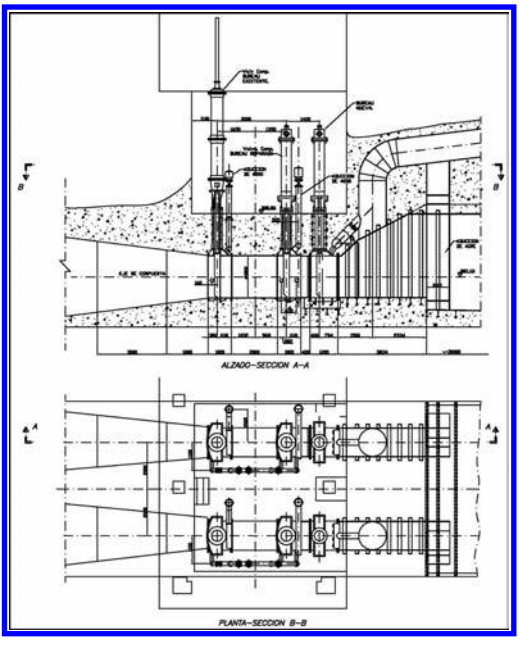

Figura 14. Fase VIII. Instalaciones accesorias.

Figura 15. Vista de la cámara de válvulas del desagüe de fondo.

Figura 16. Vista de los cierres de seguridad y regulación del conducto derecho del desagüe de fondo.

instalaciones, renovando el sistema de aireación y los by-passes.

Así mismo se terminaron de realizar las conexiones de los accionamientos al grupo oleohidráulico doble motor-bomba de 10 C.V. de inyección de aceite para el accionamiento de las 4 compuertas tipo Bureau de cada túnel, mediante tuberías de acero inoxidable para enlace con los servomotores. Para el accionamiento de las compuertas de seguridad y regulación se dispone cuatro grupos dos para cada uno de los túneles.

5 CONCLUSIONES

El aprovechamiento del río Cinca es muy importante para garantizar, a través del sistema Grado-Mediano y el canal del Cinca y junto con el aprovechamiento del río Gállego, los suministros necesarios para un sistema tan importante como el de Riegos del Alto Aragón, que abastece a 126.000 Has de regadíos, 114 núcleos de población y 765 explotaciones ganaderas a lo largo de 2.500 km^2.

Los embalses de Mediano y El Grado están entre los de mayor capacidad de la cuenca del Ebro, y con la ejecución de las obras de en los desagües de fondo de la presa de El Grado y las actuaciones planteadas en la redacción del proyecto de adecuación al RTSPE de la presa de Mediano, se logra una mejora en la funcionalidad de las dos presas, consiguiendo una mejor adaptación al reglamento vigente.

De esta manera, se consigue proporcionar una mayor garantía en el suministro, con máxima seguridad en la explotación y en la laminación de posibles avenidas.

Dam Maintenance and Rehabilitation II – Romeo García et al. (eds)
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Proposing a simple/secure structure to replace and extend the lifetime of a small/old dam spillway

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ABSTRACT: In this study, based on previous experimental data by the authors, a quantitative comparison is made between the hydraulic jump over a multistepped floor rise (w) and the free ($w = 0$) hydraulic jump, within an inclined (angle φ, with $0° \le \varphi \le 8°$) rectangular open channel. The compared quantities are, the corresponding conjugate depths' ratios, the dimensionless lengths' ratios, the water free surface profiles, the dimensionless force coefficients on both flow cases and finally, the energy losses' ratios. The results of this study may be useful for the hydraulic engineer when designing to replace and extend the lifetime of a small/old dam spillway.

1 INTRODUCTION

In small or large dams the outlet works are mainly referring to the stilling basin, which is usually a complicated (chute and floor blocks, dentated end sill) horizontal structure at the end of an inclined (angle φ) rectangular open channel of initial supercritical flow, suitable to stabilize the incipient hydraulic jump in a proper place. The body and secondary structures of the stilling basin are working very hard against the turbulent pressure fluctuations, the basin overflows or—in winter—the ice cover.

When dealing with a small—old dam and deteriorated stilling basin, it is often of primary concern to provide some remedial measures or a full replacement of it in order to extend its long term performance, of course on the presumption that the dam's main body and reservoir are still well operating.

In this study a simple, secure and economic structure is experimentally investigated to be constructed just before the end of the chute channel, in order to create an inclined jump and to result to a similar or even better functional success.

Figure 1 shows a rather complicate hydraulic jump which is part of a more general water flow phenomenon. Beyond the uniform flow cross section 1 (h_1 depth), with a supercritical flow regime described by the Froude number (q = discharge per unit channel width)

$$Fr_1 = q \cdot g^{-1/2} \cdot h_1^{-3/2} (>1) \,,$$

pthe water free surface profile is ascending to a maximum depth, h_m, at the nonuniform section m, because of a multistepped floor rise (w) which is shifting the flow over a higher channel bed, followed by a descending water free surface profile—until the (uniform flow) cross section 2 with $d_2 = h_2 - w$ actual depth. The flow between sections 1 (small depth h_1) and m (with the biggest total depth, $h_m = d_m + w$) has an external resemblance to the inclined free water jump, since the ascending water free surface profile is of similar shape to the latter hydraulic jump (Figure 2), while h_m may considered as the conjugate depth of h_1. This jump may called a "multistepped—floor/rise jump" or, shortly, a "Multistepped Jump" (M.J.). The water free surface profile may elaborated in dimensionless terms of the form

$$\overline{h} = (h - h_1)/(h_m - h_1) \text{ vs } \overline{x} = x/x_m \,,$$

where $\overline{h}$ is a function of $\overline{x}$, angle φ and (indirectly) the Froude number Fr_1.

Both hydraulic jumps are examined here within an inclined (angle φ, slope $J_0 = \sin\varphi$) rectangular open channel, where in the M.J. the front face of the floor rise consists of four equal steps (risers, treads), α, and the total rise height is $W = 4 \cdot \alpha$ (Figure 1b). From this structure a force N_x, from the front rise's face and another (shear) force F_x, from the channel boundaries, are exerted contrary to the x flow direction (Figure 1a). The water ascending free surface profile has an equation $\overline{h} = \overline{h}(\overline{x}, \varphi)$, the jump length is x_m, while the descending profile has a length x'_m.

The free inclined jump (F.J., $w = 0$, Figure 2) is developed between cross sections 1 (h'_1) and 2 (h'_m), it has a length L', the water free surface profile is $\overline{h}' = \overline{h}'(\overline{x}', \varphi)$ the channel inclination angle is φ, Fr_1' (h'_1) are larger than one, while the shear force from channel boundaries is F'_x. The free jump may often

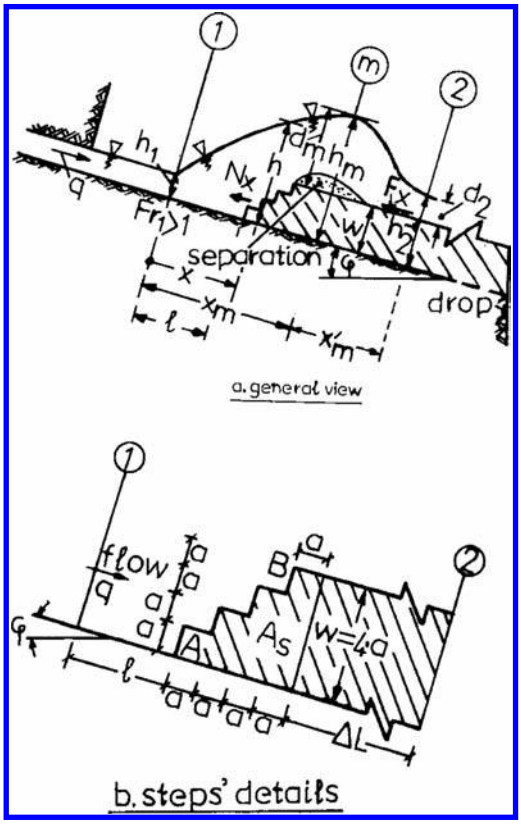

Figure 1. Multistepped jump.

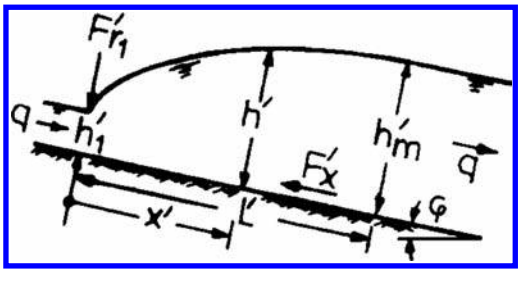

Figure 2. Free hydraulic jump.

serve as a reference hydraulic jump—when compared to other hydraulic jumps, while the water free surface profile may elaborated in terms of $\bar{h}' = (h' - h'_1)/(h'_m - h'_1)$ vs $\bar{x}' = x'/L'$.

In the present study the ratios $C^* = (h_m/h_1)/(h'_m/h'_1)$, $L^* = [\{(x_m - w)/h_1\}/(L'/h'_1)]$, and the water free surface profiles in terms of $(\bar{h})^* = \bar{h}/\bar{h}'$ vs $\bar{x}$ (or $\bar{x}'$) are examined, compared and discussed.

Furthermore, if for the multistepped jump the total pressure force (sections 1 and 2) is P_x, and for the free jump this force (at h'_1 and h'_m) is P'_x, the pertinent dimensionless force coefficients $K_G = (N_x + F_x)/|P_x|$ and $K'_G = F'_x/|P'_x|$ may compared, in the ratio $K^*_G = K_G/K'_G$. If F_x is very small compared to N_x then $K_G \cong N_x/|P_x|$. Finally, corresponding energy losses $\Delta H/H_1$ and $\Delta H'/H'_1$ for both jumps are compared in the form of the ratio $(\Delta H/H_1)^* = (\Delta H/H_1)/(\Delta H'/H'_1)$.

It should be noted again that C^*, L^*, and the water free surface profiles $(\bar{h})^*$, are compared along x_m and L' correspondingly, while K^*_G and $(\Delta H/H_1)^*$ are also compared—but K_G and $\Delta H/H_1$ (for the multistepped jump) refer to $x_m + x'_m$, while K'_G and $\Delta H'/H'_1$ refer to L' (for the free jump).

2 PREVIOUS DATA

2.1 *Free jumps*

Chow's book (1959) consists the most well known reference for free jumps, while Visher et al (1995) present a wide summary—mainly—of horizontal hydraulic jumps. Demetriou (2005) is presenting a large number of measurements on the inclined free hydraulic jumps for $0° \le \varphi \le 16°$ and F'_{r1} up to 19. This author gives also suitable equations for the water free surface profiles and the ratios h'_m/h'_1, L'/h'_1. As far as the shear force (F'_x) in inclined free jumps is concerned, Demetriou (2008, a, b, c), and (2009), is presenting some pertinent indirect results based on the onedimensional momentum equation. For other specific topics in inclined jumps Demetriou et al (2008, a, b), are also giving some results calculated after h'_1, h'_m, L', φ, and $F'r_1$ measurements. Finally, Dimitriou et al (unpubl. a, b, c, d, e) give some experimental results on inclined jumps over a multistepped rise.

2.2 *Multistepped jumps*

In the last unpublished five papers the authors are giving some indirect (based on various h_1, h_2, X_m, x'_m, φ and Fr_1 experimental measurements) results on the mechanical energy losses. Similarly, $N_x + F_x$ forces on inclined multistepped jumps are indirectly calculated, based on the water weight under the free surface profiles and the onedimensional momentum equation. All laboratory measurements are extended to $0° \le \varphi \le 8°$, $3 \le Fr_1 \le 11$ and $0.1 \le h_1/W \le 1.3$.

All these previous data are used here for a comparison of pertinent quantities between inclined (or horizontal) free and multistepped jumps. For each comparison the corresponding previous diagrams are presented, followed by C^*, L^*, free surface profiles $(\bar{h})^*$, K^*_G and $(\Delta H/H_1)^*$ ratios' calculations and

new diagrams, in order to quantitively show the comparisons' results. Any comparison is made for $Fr_1 = Fr_1'$, or same q and $h_1 = h_1'$. This means that $C^* = h_m/h_m'$, and $L^* = (x_m - w)L'$, where for the free jump it is $w = 0$ $(L' - w = L')$.

3 COMPARISON. ANALYSIS AND DISCUSION

Figure 3a shows the conjugate depths ratio h_m/h_1 vs Fr_1 and angle φ for the multistepped jump (M.J.) and Figure 3b presents the same ratio h_m'/h_1' vs Fr_1 and angle φ for the free jump (F.J., $w = 0$) while Figure 4 presents the ratio C^* vs Fr_1 and angle φ.

As one can see from Figures 3a and 3b both h_m/h_1 and h_m'/h_1' vs Fr_1 and angles φ are increasing with angle φ, probably because of the water weight component which is present in inclined jumps.

The comparison of h_m/h_1 and h_m'/h_1' (or h_m/h_m' for $h_1 = \bar{h}'$) in their ratio C^*, which is presented

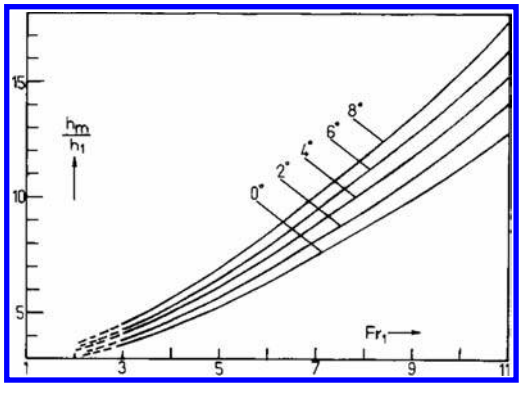

Figure 3a. h_m/h_1 vs Fr_1 for M.J. and $0° \le \varphi \le 8°$.

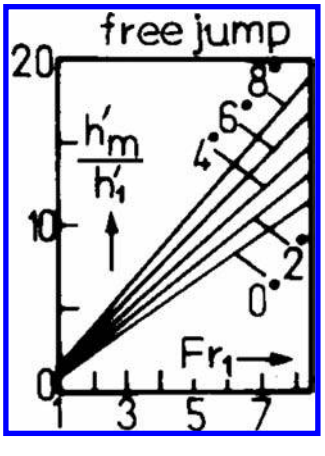

Figure 3b. h_m'/h_1 vs Fr_1 for F.J. and $0° \le \varphi \le 8°$.

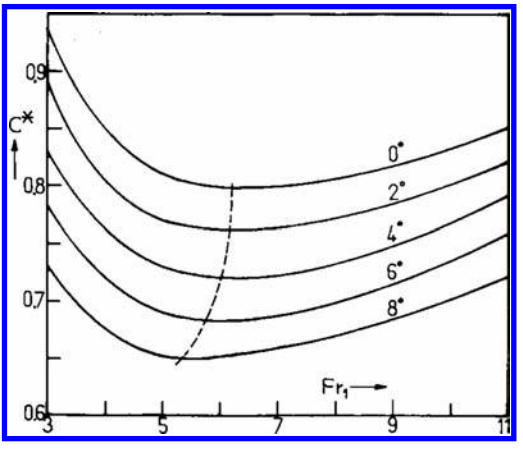

Figure 4. C^* vs Fr_1 and angle φ for M.J. and F.J.

in Figure 4 (vs Fr_1 and angles φ) shows, that in the present fields of parameters, it is $C^* < 1$, i.e. $h_m/h_1 < h_m'/h_1'$, and that—although the rise (w)—this inequality becomes stronger when angle φ is increasing. Moreover, any of the curves shown in Figure 4 has a falling and a rising branch, i.e. the same C^* may be achieved for two different Fr_1 values—along angle φ = const. For example the value $C^* = 0.75$ for $\varphi = 4°$ holds for $Fr_1 \cong 4.3$ and $Fr_1 \cong 9$, while all minimum C^* values approximately appear to lie on the broken curve of this figure. To any of these minimum C^* ratios corresponds only one Fr_1 value, for example for $\varphi = 4°$, $C^* \cong 0.725$ for $Fr_1 \cong 6$.

Figures 5a and 5b present $(x_m - w)/h_1$ and L'/h_1' (vs Fr_1 and angle φ) for the M.J. and F.J. $(w = 0)$ correspondingly. All pertinent curves appear to be systematic, where for Fr_1 = const. $(x_m - w)/h_1$ and L'/h_1' are increasing with angle φ.

Figure 6 shows the pertinent ratios $L^* = [\{(x_m - w)/h_1\}/(L'/h_1')]$ vs Fr_1 and angle φ, where for Fr_1 = const. and $h_1 = h_1'$, $L^* = (x_m - w)/L'$. This figure shows that, in the present field of parameters, it is $L^* < 1$, or $[(x_m - w)/h_1] < (L'/h_1')$ or, for $h_1 = h_1'$ it is $(x_m - w) < L'$, and that this inequality becomes stronger when angle φ is decreasing.

Figures 4 and 6 show that in general, the dimensionless conjugate depths and lengths for M.J. are less than corresponding quantities for F.J. $(w = 0)$. This depicts the fact that a structure like a multistepped rise creates a jump of smaller relative total depths and lengths, which in some cases may offer some design advantages.

Figures 7a and 7b present the water free surface profiles for the multistepped and the free jump correspondingly, in terms of $\bar{h}$ vs $\bar{x}$ and $\bar{h}'$ vs $\bar{x}'$. The results are systematic and show that for $\bar{x}$ = const. (or $\bar{x}'$ = const.) $\bar{h}$ (or $\bar{h}'$) are decreasing with angle φ increase.

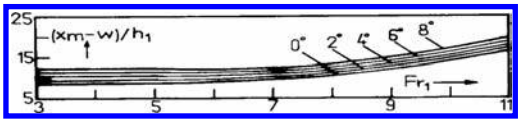

Figure 5a. $(x_m - w)/h_1$ vs Fr_1 for M.J. and $0° \le \varphi \le 8°$.

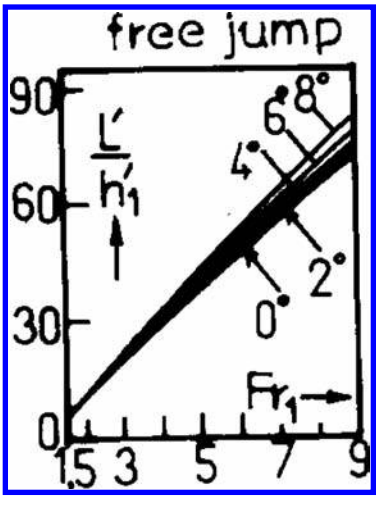

Figure 5b. L'/h'_1 vs Fr_1 for F.J. and $0° \le \varphi \le 8°$.

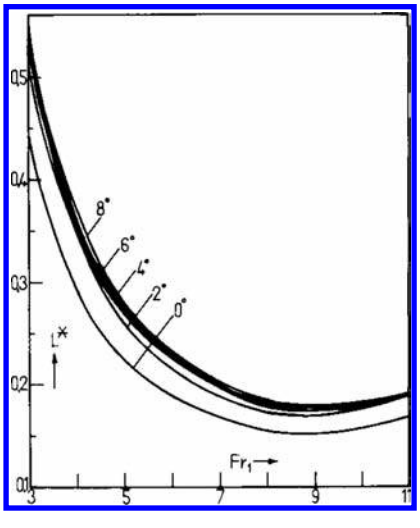

Figure 6. L^* vs Fr_1 and angle φ for M.J. and F.J.

Figure 8 shows the comparison of $\bar{h}/\bar{h}' = (\bar{h})^*$ vs $\bar{x}$ (or $\bar{x}'$), where, apart from $\bar{x} = 1$ or $\bar{x}' = 1$, it is $(\bar{h})^* < 1$, i.e. $\bar{h} < \bar{h}'$: The water free surface profiles, between $\bar{x} = 0$ (or $\bar{x}' = 0$) and $\bar{x} = 1$ (or $\bar{x}' = 1$), of the multistepped jump are lower than corresponding water free surface profiles of the free jump.

Figure 8 shows also that for about $\bar{x} > 0.25$ (or $\bar{x}' > 0.25$) all $(\bar{h})^*$ results fall on a single line, while

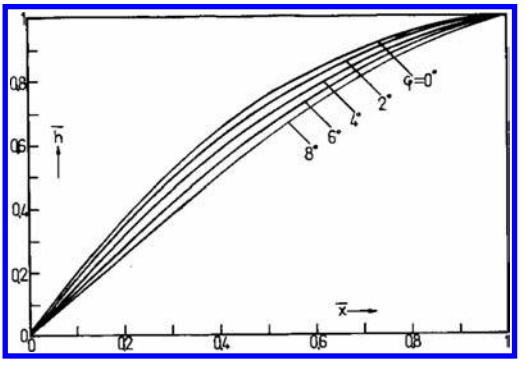

Figure 7a. Profiles for M.J. and $0° \le \varphi \le 8°$.

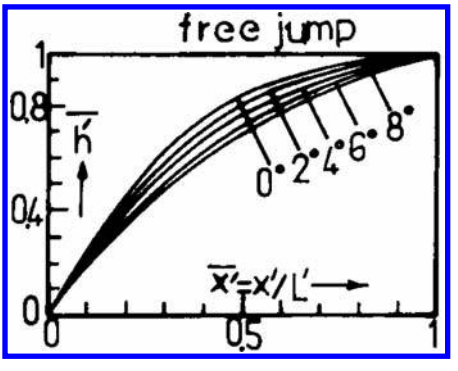

Figure 7b. Profiles for F.J. and $0° \le \varphi \le 8°$.

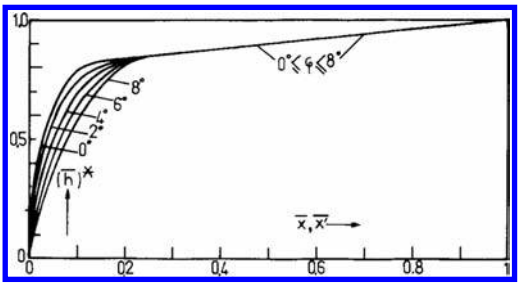

Figure 8. Comparison of profiles for M.J. and F.J.

under this limiting $\bar{x}$ (or $\bar{x}'$) value corresponding curves are separate (but systematic) among them.

Figure 9a shows the force coefficients K_G vs Fr_1 and angles φ for the multistepped jump, while Figure 9b shows the coefficients K'_G for the free jump, where the latter coefficients are very small in front of K_G. For example for $\varphi = 6°$ and $Fr_1 = 8$ it is $K_G \cong 57.5$ (Figure 9a), while $K'_G \cong 0.125$, that is to say $K^*_G \cong 460$. This big difference is due to the strike of the supercritical flow on the multistepped front face of the rise in the case of M.J., while the F.J.

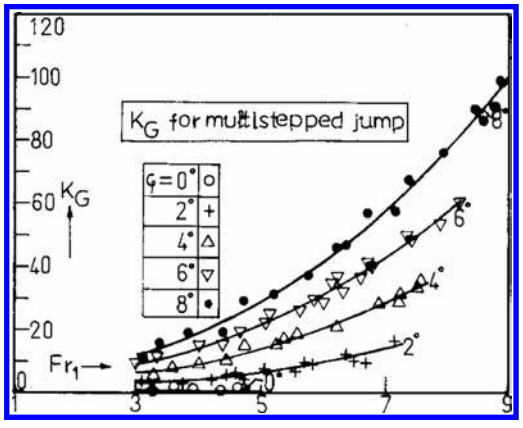

Figure 9a. K_G for M.J. and $0° \le \varphi \le 8°$.

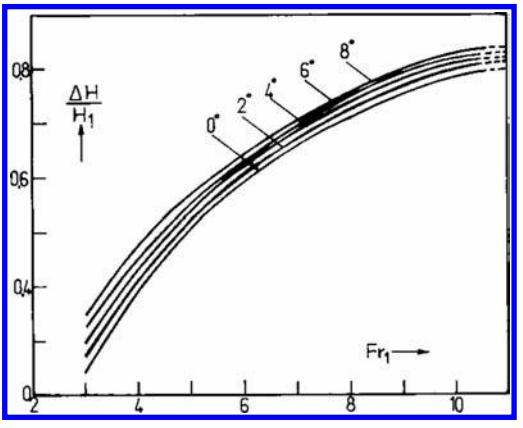

Figure 10a. $\Delta H/H_1$ vs Fr_1 and $0° \le \varphi \le 8°$ for M.J.

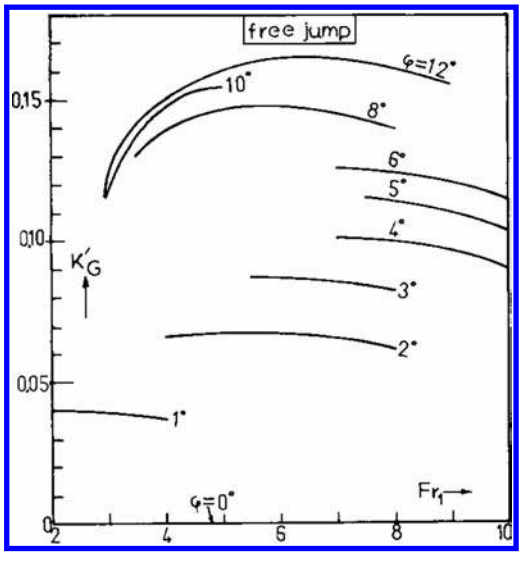

Figure 9b. K'_G for F.J. and $0° \le \varphi \le 12°$.

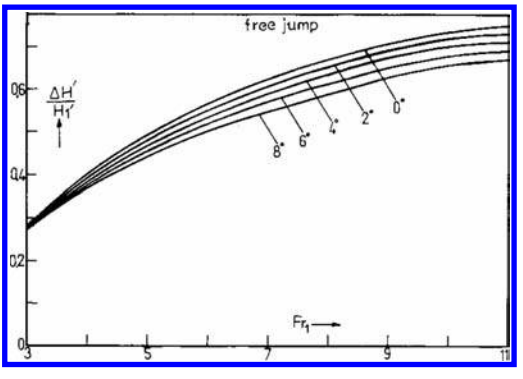

Figure 10b. $\Delta H'/H'_1$ vs Fr_1 and $0° \le \varphi \le 8°$ for F.J.

appears to have corresponding force F'_x, which—as only a boundary shear force—is very small.

Since there is a big difference in K_G and K'_G coefficients there is no need to construct a diagram of K^*_G vs Fr_1 and angle φ.

Figures 10a and 10b present the dimensionless mechanical energy losses $\Delta H/H_1$ (M.J.) and $\Delta H'/H'_1$ (F.J.) vs Fr_1 and angles φ. $\Delta H/H_1$ appears to systematically increase with Fr_1 and angle φ, while $\Delta H'/H'_1$ show a small change for angles $0° \le \varphi \le 8°$ within the range $3 \le Fr_1 \le 11$, and—as also noted by Visher et al (1995)—$\Delta H'/H'$ are decreasing when angle φ is increasing. This fact shows that M.J. may successfully be used to increase the energy losses when this is necessary.

Figure 11 presents $(\Delta H/H_1)^* = (\Delta H/H_1)/(\Delta H'/H'_1)$ ratios (vs Fr_1) for a comparison between M.J. and F.J. from the point of view of energy losses in the fields $3° \le Fr_1 \le 8°$ and $0° \le \varphi \le 8°$, where the comparison is presented only for the two limiting angles $\varphi = 0°$ and $\varphi = 8°$. As it may be seen from Figure 11, in these fields of parameters—apart from a limited small Fr_1 values range for $\varphi = 0°$—it is $(\Delta H/H_1)^* > 1$, or $(\Delta H/H_1) > (\Delta H/H'_1)$, i.e. the dimensionless energy losses for the M.J. are bigger than corresponding losses for the F.J. This fact is due to the presence of the floor rise in the case of the M.J., and as seen in Figure 11, when angle φ is increasing towards $\varphi = 8°$, $(\Delta H/H_1)^*$ are increasing too, while both lines appear to achieve some liming values for larger Fr_1.

As a final remark it can be stated that the geometrical elements (conjugate depths, lengths and water free surface profiles) of the multistepped jump are smaller than corresponding elements of the free jump, while force and energy losses' elements appear to be larger in multistepped jumps than in free jumps.

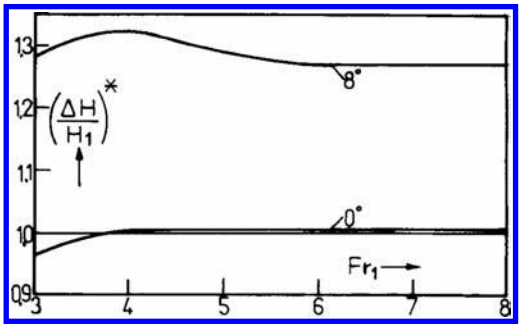

Figure 11. Comparison of $(\Delta H/H_1)^*$ ratios vs Fr_1 and φ for M.J. and F.J.

3 CONCLUSIONS

In this study, based on previous experimental data by the authors, a comparison is made between a group of inclined (angle φ) jumps over a multi-stepped floor rise w (M.J.) and a group of inclined free jumps, $w = 0$ (F.J.), in the fields of $3° \le Fr_1 \le 8°$ and $0° \le \varphi \le 8°$. The main results are: 1) The corresponding conjugate depth ratios of the M.J. are smaller than of the F.J. ratios (for same angle and Fr_1). 2) Corresponding dimensionless lengths show a similar behavior. 3) Corresponding water free surface profiles are lower in M.J. than in F.J. 4) Corresponding dimensionless resisting force coefficients are much bigger in the case of M.J. than in the case of F.J. 5) Corresponding dimensionless energy losses are higher enough for M.J.—than for F.J. The results of this study may be useful for the hydraulic engineer when designing hydraulic structures along the outlet works of small dams.

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Numerical modeling of two phase flow in Morning-Glory spillways

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ABSTRACT: In this paper, the optimum inlet shape of morning glory spillway is discussed. The effect of some parameters including maximum discharge and minimum possibility of cavitation are investigated. For this purpose, using Computational Fluid Aynamics (CFD) in the form of finite volume method, 3D Navier-Stokes Equations of flow at different inlet shapes are solved. Free water surface is modeled by water-air two phase flow and equations are solved by VOF method. Flow turbulence is modeled by "K-Є model". Based on experimental results, models are verified and discharge, velocity, pressure and cavitation index for different inlet shapes are computed and compared. The Morning-Glory is modeled using different types of equations including WAGNER equation ($Y = aX^{3.88}$), CREAGER equation ($Y = aX^{1.87}$) and circular form for their funnel profile shapes. In each model, head (h) versus discharge (Q) diagram is drawn and results are compared.

1 INTRODUCTION

1.1 *Morning-Glory spillway*

Morning-Glory spillway is one of the common spillways for discharging the overflow water behind dams; these kinds of spillways are constructed mostly in reservoirs of low-volume dams and canyons. The most important advantage of Morning-Glory spillway is its less construction costs by using diversion tunnel constructed at the time of dam construction as a horizontal tunnel.

Morning-Glory spillway has three main parts: Cup-shape inlet funnel, vertical shaft, and relatively horizontal diversion shaft (In some cases a quiet basin is constructed at the end of horizontal tunnel). The designing discharge should develop the free flow in the shaft; that is the capacity of vertical shaft and diversion shaft should be more than the capacity of spillway inlet structure. An aerator is contrived in vertical shaft in order to have the atmosphere pressure all along the spillway up to the place of energy dissipation. The spillway usage is recommended when:

- The earthquake probability is low.
- The horizontal spillway can be connected to the current diversion shaft.
- The rate of trash and floating materials are not significant.
- There is a space limitation for constructing the direct spillway.
- The geotechnical conditions in regard to structural settlement is suitable and
- A short diversion shaft can be prepared.

1.2 *Types of flow state in Morning-Glory*

Morning-Glory spillway may be designed as "Free flow" or "Submerged". In free flow state it acts like normal direct spillway and the Discharge have a relationship with the 3/2 power of the head on top of the spillway, but in submerged state the flow conditions are completely different, Discharge changes have a relationship with the square root of the depth on top of the spillway; in this situation when the water height on top of the spillway increases, the discharge increment will be very low. Thus if spillway is designed as submerged, a very high safety factor should be considered for its capacity.

Different states of flow in Morning-Glory spillway have been shown in Figure 1. As it is seen in the chart, in the first state (Crest control) the flow discharge is from point "a" to point "g", in the second state (Tube control) the discharge is from point "g" to point "h", and in the third state (Conduit control) the discharge is from point "h" upwards. Each of the above mentioned states may occur by varying spillway's dimensions and the head on the spillway.

2 NUMERICAL MODEL OF MORNING-GLORY SPILLWAY

Because of relative complex geometry of models, irregular meshes including a combination of rectangular and triangular meshes have been used and the geometry of studied models have been

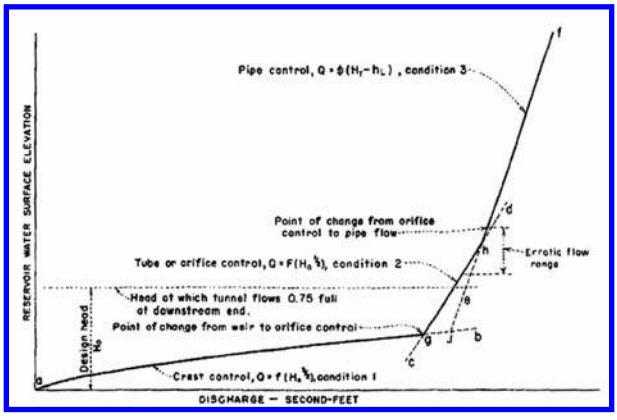

Figure 1. Different states of flow in Morning-Glory spillways.

practiced as three-dimensional and with different meshes.

The number of used cells in this mesh was as follows: first, the meshes with multimillion cells have been used for the model. It was remarked that in comparison with the mesh having multi-thousand cells, there is a small difference among them regarding the specifications of flow. Therefore because of high number of considered exercises and their time-consuming specification, it was decided to use multi-thousand mesh and the finer meshes are merely used for comparison.

Because of high capability in modeling the flow in the state of free and under-pressure flows combination, the finite-volume FLUENT software was used for stimulation of spillways. Fluid volume model was applied because of its capability in forming the free surface and because of complexity of flow and presence of vortex, three dimensional models was preferred to use. Also, flow turbulences were modeled in the form of both "k-ε" and "Reynolds stress" models, and after comparison, "k-ε" model was used because of its simplicity, its usage in similar works and also its prevalence. Two-phase flow including water and air has been used for flow stimulation. The followings have been considered for density and viscosity of water and air:

Water: P = 1000 Kg/m^3 μ = 0.001 Kg/m.s
Air: P = 1.225 Kg/m^3 μ = 1.79 e-5 Kg/m.s

A concentric cylinder was used on the spillway in order to develop a reservoir head on the spillway, and generally the boundary conditions are as Tables 1–5.

The head on the reservoir was considered from 0 to 50 meters in 10 stages with 0.5 meters variations. Discharge, cavitation index and other specifications of the flow have been studied in each stage.

Table 1. The boundary conditions of numerical model.

Model inlet boundry	Pressure-inlet
Outlet of spillway	Pressure-outlet
Walls of spillway	Wall

It should be mentioned that vortex phenomenon has not been considered in the first stage, because the reservoir's head in this stage is less than submerged limit, so that only the inlet funnel form can determine the rate of discharge (Crest control).

As it was mentioned, three different spillways with different inlets were modeled, and each has been studied for different heads less than the submerged depth.

2.1 *Model 1 (CIRCULAR Model)*

2.1.1 *Geometry and meshing of model 1*

The geometry of model 1 is shown in Figure 2. This spillway consists of three Parts:

1. Inlet orifice with 13 meters initial diameter and 8 meters final diameter; this cross section variation in profile follows the circular equation.
2. Prismatic vertical shaft with 44.66 meters height.
3. 90 degree bending with 8 meters diameter.

Irregular combined mesh has been used. The number of used cells is 214483, Mesh Grid and boundary conditions are shown in Figure 3.

2.1.2 *The results of model 1*

After making the geometry, meshing and modeling the spillway for 10 different depths, the rate of discharge and flow specifications were studied in each stage. Figure 4 shows the spillway's vertical section (Y = 0).

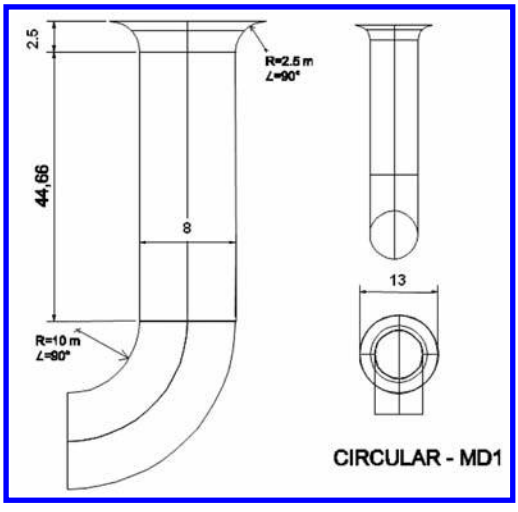

Figure 2. Geometrical specifications of model 1.

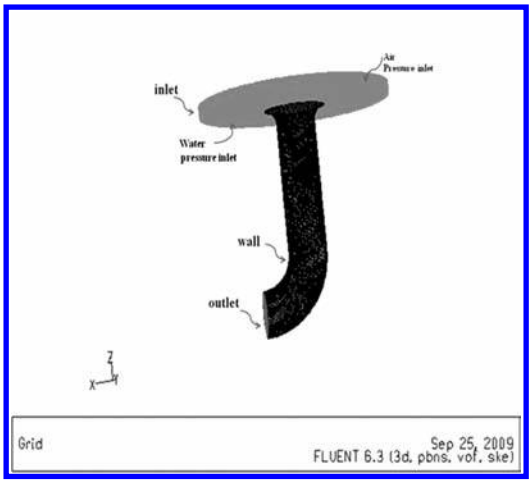

Figure 3. Meshing & boundary conditions of model 1.

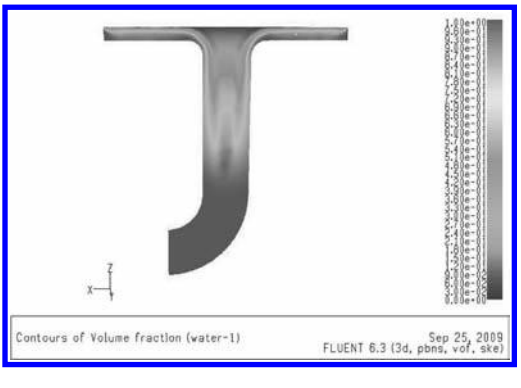

Figure 4. Vertical section of model 1 for h = 2 m.

Figure 5 shows the chart for the relationship of discharge variations to the head on spillway.

Figure 6 also shows the chart for relation of discharge coefficient to h/R.

Since three modeled spillways are different in scales because of special geometric form, discharge coefficient parameter versus dimensionless number is used for comparing the reservoir's discharge capability, discharge coefficient in Crest control state is:

$$C = \frac{Q}{Lh^{\frac{3}{2}}} \tag{1}$$

where h is the head on the spillway, R is the discharge inlet radius, and L is the environment of the spillway's external edging.

As it can be seen in Figure 7, the range is from 0.15 to 0.55. The reason for selecting this range is

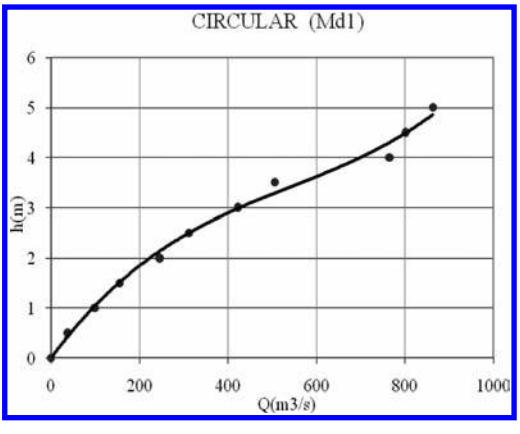

Figure 5. Relation of discharge variations to the head for model 1.

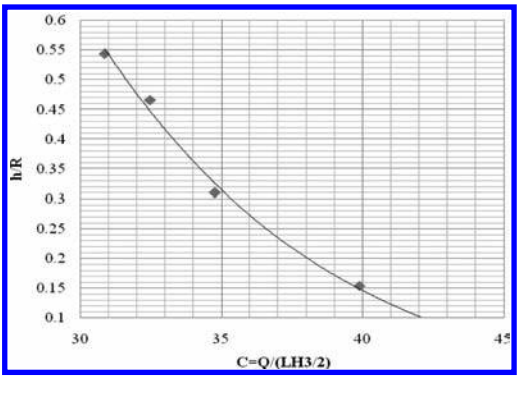

Figure 6. Chart for relation of discharge coefficient to h/R for model 1.

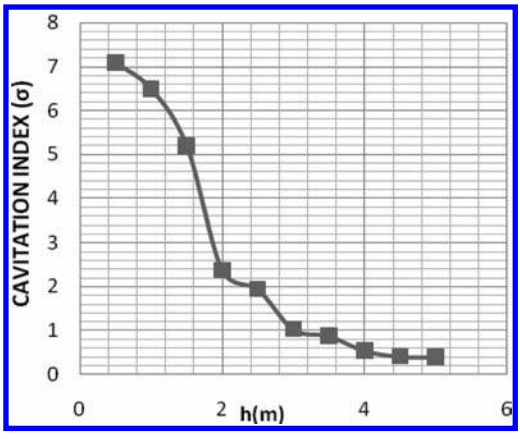

Figure 7. Cavitation index distribution on spillway frame foe model 1, for 2 meters head.

that because of the adherence between water and spillway's wall under 0.15 in this model, flow's profile would become unstable and any value more than 0.55 is more than Crest control.

As it was mentioned before, one of the important factors in designing Morning-Glory spillway is the possibility of destructive cavitation phenomenon occurrence. Cavitation index is the most important tool used for cavitation estimation, which is obtained by the following equation:

$$\sigma = \frac{P - P_v}{\frac{\rho V^2}{2}} \qquad (2)$$

P and P_v are flow absolute pressure and steam pressure, respectively, and V is the velocity in each point of flow.

Figure 7 shows the cavitation index distribution on spillway's frame in model 1 for 2 meter head. In this state the velocity in all spots of the model is less than 9 meter per second, in which the cavitation index is always more than 0.25 critical rate, and the possibility of cavitation occurrence is very small.

Based on performed surveys in model 1, the point which is considered as the critical for cavitation in spillway's inlet section is located in the junction of inlet funnel and vertical shaft in which there is a possibility of flow separation.

2.2 *Model 2 (WAGNER Model)*

2.2.1 *Geometry and meshing of model 2*

The geometry of model 2 is shown in Figure 8. This model also consists of three main parts:

1. Inlet orifice which follows the Wagner function: $y = 0.0102x^{3.818}$
2. Prismatic vertical shaft with 8.93 meters length and 7 meters diameter.
3. 90 degree bending with 19.73 meters central radius and 7 meters diameter.

Meshing of Wagner model has 369571 cells, and as the previous model, irregular mesh has been used. After creating initial and boundary conditions, the spillway was examined for 10 different depths, and then the results were studied.

2.2.2 *The results of model 2 (WAGNER)*

Figure 9 shows discharge variations chart for different heads on model 2.

Figure 10 shows the relation of discharge coefficient variations to h/R for Wagner spillway.

As it can be seen by studying model 2, the most critical point for cavitation occurrence in the funnel section is in a point between 0 and 1 in Figure 8, which follows the Wagner function. The relation of cavitation index variations to the head for the most critical point in model 2 has been shown in Figure 11.

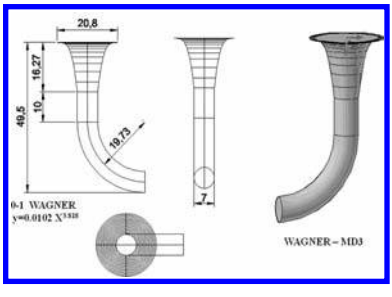

Figure 8. Geometrical specifications of model 2.

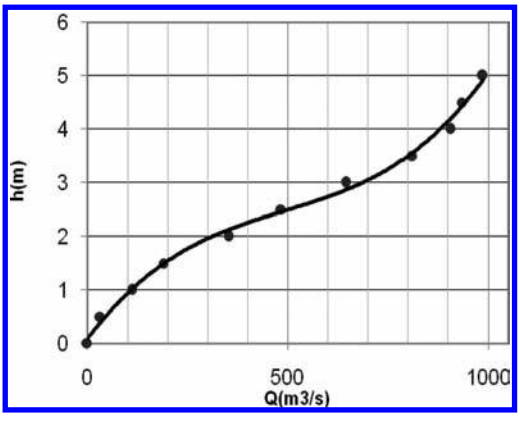

Figure 9. The chart for relation of discharge variations to head for model 3.

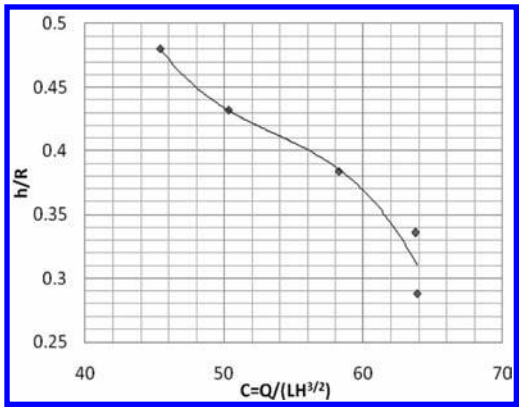

Figure 10. Relation of discharge coefficient variations to h/R for spillway in model 2.

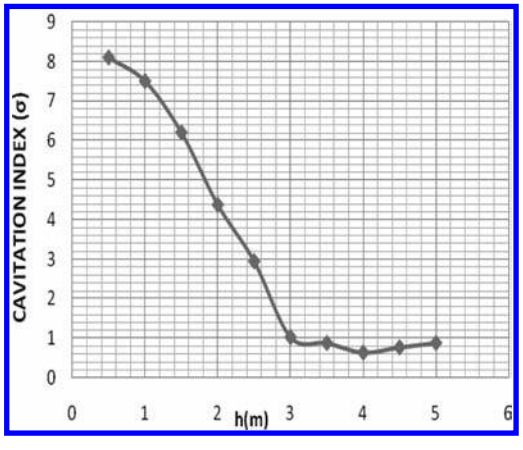

Figure 11. Relation of cavitation index variations to the head for the most critical point in model 10.

2.3 *Model 3 (Creager)*

2.3.1 *Geometry and meshing of model 3*

Figure 12 shows the geometry of model 3. This spillway consists of three parts:

1. Inlet orifice which is a funnel with creager equation, and initial radius of 9.68.
2. Non-prismatic vertical shaft with section linear variation, which begins by 8.52 meters radius and 8 meters length, and ends in 7.12 meters radius.
3. Non-prismatic 90 degree bending with section linear variation which begins by 7.12 meters radius and ends in 2.25 meters.

As it is shown in figure 12, the inlet curve in this model is less than three other models, furthermore, the flow on circular spillway can be seen by comparing the figure, and the funnel curve similar to flow profile on it is high in relative heads (h/R). So as it is clear in Figure 13, comparing to previous models this spillway shows less variation in discharge coefficient by increasing the head. High ascent in top of the chart in Figure 13 confirms this action.

2.3.2 *The results of model 3 (CREAGER)*

Study of this model for different heads illustrates that the most critical point for spillway is the conjunction point of non-prismatic vertical shaft to the bending in the tunnel.

In a chart shown in Figure 14 the relation of discharge coefficient to depth for model 3 can be seen. Conclusion is that this spillway shows a more steady action to the head's variations, whereas more ascent of this chart means that by increasing the head, the spillway would have more capability in discharging the reservoir. This proper action of spillway can be seen in discharge coefficient chart.

Figure 15 also shows the relation of cavitation index to the relative depth (h/R) for this model.

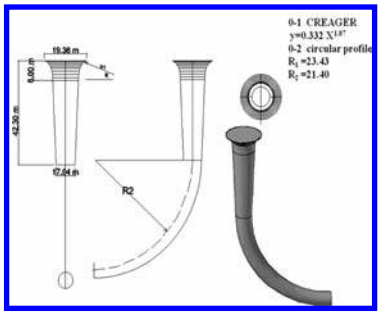

Figure 12. Geometrical specifications of model 3.

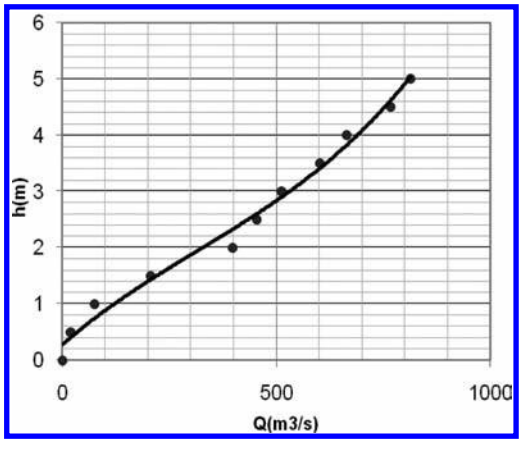

Figure 13. Chart for relation of discharge variations to the head in model 3.

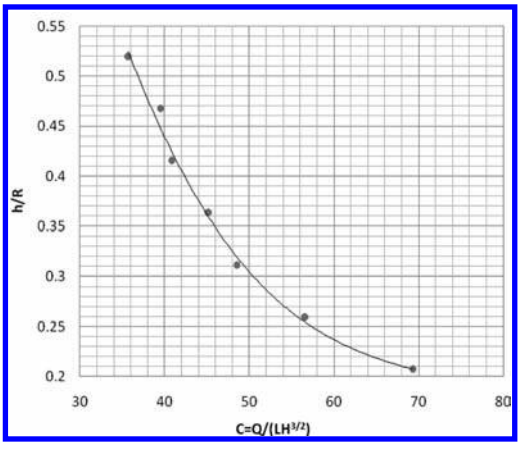

Figure 14. Relations of discharge coefficient variations to h/R for spillway in model 3.

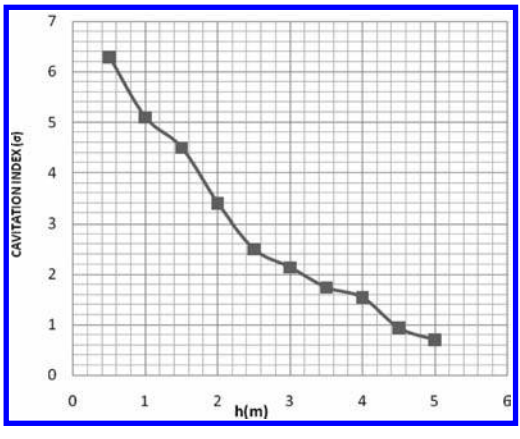

Figure15. Relation of cavitation index variations to the head for the most critical point in model.

3 CONCLUSIONS

3.1 *The study of discharge-head chart*

A chart shown in Figure 16 illustrates the relation of discharged rate of all three models to the head. As it is shown each of three curves consists of three parts:

1. The first part with a great ascent relates to the heads in which the spillway's inlet is controller.
2. The second part with moderate ascent relates to tube control state in which the spillway is in semi-submerged status.
3. In the curve's third part which has a great ascent, the spillway is in completely submerged state and the spillway's bending is in under-pressure flow state.

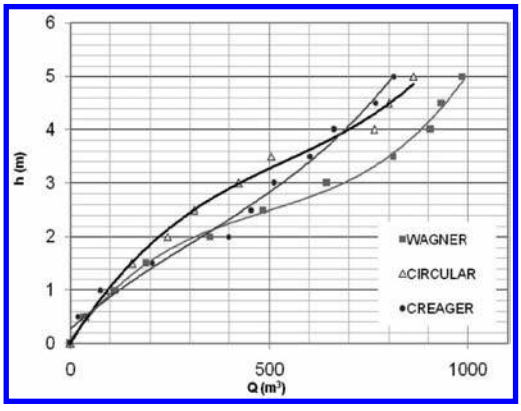

Figure 16. Relation of discharged rate to the head for three models.

The curve in model 1 has a great ascent in two first parts, but it has a lower ascent in the chart's final part comparing to three other models. So, it is concluded that as it was expected this spillway has a higher discharge capability in high relative heads.

The spillway in model 2 has less ascent in two first parts of its curve, but it rises in the final part. So, in this model also as it was expected the spillway for low relative heads has a great discharge capability, but once the head increases and the spillway is submerged, the rate of outlet discharge will inclined to a static rate. Model 2 and 3 have a relatively similar action, but the spillway in model 3 has a low ascent at first, while it has a greater ascent in middle part comparing to model 2. Therefore it is concluded that the spillway in model 3 has more discharge capability in Crest control state and model 2 has more discharge capability in tube control state. Since the Crest control state is important for us in comparing the spillways` inlet shape, inlet funnel in model 2 is more suitable. But because the spillway has been modeled with different dimensions, the proper tool for comparing their discharge capability is discharge coefficient, a chart for the relation of which to the relative head's variations has been shown in Figure 17.

3.2 *The study of relation of discharge coefficient to the relative head ((h/R)-C)*

According to the chart in Figure 17, model 1 has a lower discharge coefficient than three other models.

Therefore the optimal form of inlet should be selected between model 2 and 3. Discharge coefficient in model 2 is more than model 3, but the difference is very small. Also, the curve ascent in model 3 for high h/R is more. So, it can be concluded that once the head increases, discharge

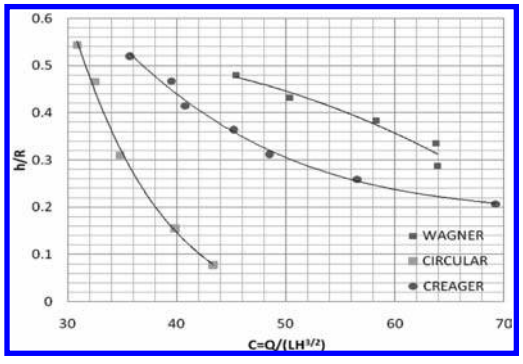

Figure 17. Relation of discharge coefficient to relative head for 3 models.

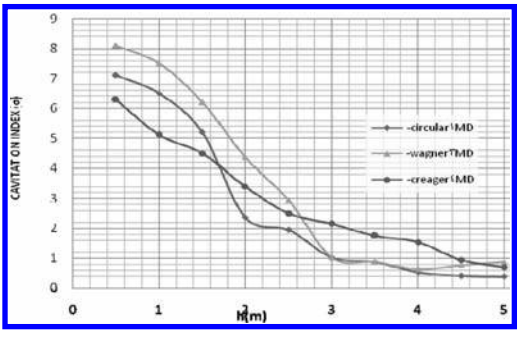

Figure 18. Relation of cavitation index to the relative head for three models.

capability of this model will be less decreased and totally it has less sensitivity to the reservoir's head. This was shown in a chart in Figure 16. i.e. a chart for relation of Q to h, and the curve in model 3 was near line and no great variations have been observed in the ascent in first part.

3.3 *The study of relation of cavitation index to the head*

Figure 18 shows a chart for relation of cavitation index to depth for the most critical point of inlet funnel for all three models. It should be mentioned that there are more critical points in bending parts of the models, but as we study the inlet shape so the critical point has been chosen from funnel section.

In model 1 and 2 the cavitation probability is higher because of high curve in the point of section change. As it is shown in the chart, in heads more than 3 meters the cavitation index of model 1 and 3 is less than 1, but in model 3 the cavitation probability is less because of the moderate curve of funnel.

Concerning the mentioned notes, it is clear that each of inlet shapes is appropriate for its own discharge and reservoir conditions. However, generally according to discharge index, model 2 and 3 are more optimal for cavitation index of economic and executive geometrical form for-reservoir's common conditions. By comparing the discharge capability of high relative heads and cavitation index of critical point, model 3 is chosen as the inlet optimal form.

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El proyecto de adecuación de la presa de Camarillas (Albacete)

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ABSTRACT: The Camarillas dam is located in the Mundo river, which is tributary of the Segura river by the left bank. It is downstream Talave dam, which receives the Tajo—Segura channel tributaries. The construction of Camarillas dam was finished in 1935, and since then has been managed with no interruption. Its facilities are degraded due to its old age and the excessively operation of the elements, demanded by its particular strategy situation. The project of modernisation and adaptation to the current Normative has been made and it includes the adaptation of the two spillways (in one of them by the project of a labyrinth weir) and also the rehabilitation of the bottom outlets with subterranean works. This project will permit to recover the reservoir capacity.

RESUMEN: La presa de Camarillas se ubica en el río Mundo, tributario del río Segura por su margen izquierda y aguas abajo de la presa de Talave, embalse que recibe las aportaciones del trasvase Tajo—Segura. Camarillas se terminó de construir en 1935 y desde entonces se ha explotado de manera continua. Las instalaciones se encuentran en un avanzado estado de degradación, debido al paso de los años y a una excesiva frecuencia de maniobra exigida por su posición estratégica. Se ha redactado el proyecto de modernización de sus instalaciones así como de adecuación a la Normativa actualmente vigente. Las obras contempladas incluyen la adecuación de los dos aliviaderos de superficie, en uno de ellos mediante la construcción de un vertedero tipo laberinto, y la rehabilitación de los dos desagües de fondo, en ambos casos mediante obra subterránea. Todo ello permitiría recuperar la capacidad de regulación original del embalse.

1 INTRODUCCIÓN

La presa de Camarillas (Albacete) se ubica en el congosto de los Almadenes, en el río Mundo, tributario del río Segura por su margen izquierda. La presa se encuentra aguas abajo de la presa de Talave, presa ésta que recibe las aportaciones del trasvase Tajo—Segura, lo que concede a Camarillas un importante papel estratégico en la regulación hídrica de todo el levante español.

El final de la construcción de la presa data de los últimos años de la década de los 30. Desde entonces la presa y sus instalaciones han permanecido ininterrumpidamente en servicio.

La antigüedad de las instalaciones y los criterios de diseño bajo los que se proyectaron, su explotación prácticamente ininterrumpida y mayor de la prevista debido a la puesta en servicio del trasvase Tajo—Segura, y el difícil acceso a los elementos de control y maniobra, hacen preceptivo acometer las actuaciones que permitan rehabilitarlas y modernizarlas.

En el año 2006 se redactó el "Proyecto para la adecuación a la normativa vigente de los aliviaderos, desagües y tomas de la presa de Camarillas" que contempla la actuación en ambos aliviaderos de superficie, hasta obtener una capacidad hidráulica tal que permita recuperar la capacidad reguladora del embalse, actualmente restringida por motivos de seguridad, la rehabilitación de los desagües de fondo, cuyo funcionamiento se encuentra comprometido, además de por el estado de sus elementos, por los niveles del río aguas abajo, y la modernización de las tomas de agua.

2 LA PRESA DE CAMARILLAS

La presa de Camarillas es de gravedad, de planta recta y fábrica de hormigón. Su altura es de 36.90 m sobre el cauce y de 41.50 m sobre cimientos. El paramento de aguas arriba es vertical y el de aguas abajo tiene talud 0.735/1 en el tramo superior

de unos 24 m de altura, (desde la intersección del castillete de coronación con el propio paramento hasta la cota 325.68 m) y de 0.85/1 en el tramo inferior, por debajo de la cota 325.68 m hasta la cimentación.

La cota de coronación, medida sobre el eje de la calzada, es la 357.36 m y el vértice teórico de la sección queda a 1 m por debajo de ésta (a la cota 356.36).

Las paredes del estrecho en que se ubica la presa son casi verticales y de una altura, sobre el lecho del río, del orden del doble de la que alcanza aquélla. Los tramos del camino de servicio inmediatos a la presa (a los que da continuidad la calzada de coronación) están excavados en túnel.

La longitud de coronación de la presa es de 32 m. La presa no se dotó de juntas transversales, ni de galerías de inspección, y por lo tanto carece de drenaje.

El cuerpo de presa, de hormigón en masa, está compuesto por una capa de sillería de 0.60 m de espesor y dos zonas con hormigón de diferentes dosificaciones, una de 200 kg/m^3 de cemento, en el cuerpo de presa, y otra de 300 kg/m^3 de cemento en una franja de 2 m de espesor del paramento de aguas arriba. El volumen total de hormigón en el cuerpo de presa es de 15920 m^3.

El aliviadero derecho del embalse de Camarillas responde a una morfología común en las presas de la cuenca del río Segura (presa de Alfonso XIII, por ejemplo): Dos vanos regulados por compuertas

Figura 1. La presa de Camarillas. Fotografía del paramento de aguas arriba.

Figura 2. Fotografía del aliviadero derecho.

de sector circular que evacuan los caudales directamente en una rápida que enlaza con la descarga, en planta, mediante los cajeros que, simétricamente respecto del eje del aliviadero, convergen hasta adoptar la anchura del canal de descarga, que a su vez atraviesa la ladera, en túnel, hasta restituir los caudales al cauce fluvial.

El aliviadero izquierdo atraviesa en túnel la ladera del mismo lado. La embocadura está conformada por un vano de 5.75 m de longitud y regulado por una compuerta STONEY y por otro de labio fijo con disposición circular en planta.

El vano regulado por la compuerta STONEY está delimitado por una pila por su parte derecha, que lo separa del vano de labio fijo, y por el estribo del aliviadero en su parte izquierda. Ambas estructuras (pila y estribo) soportan un puente sobre el vano desde el cual se maniobra la compuerta STONEY.

El vano de labio fijo del aliviadero tiene una disposición de arco de círculo en planta de unos 16.56 m de radio, con una longitud total de 23.56 m. La cota del umbral se encuentra a la 354.63 m.

Los dos vanos conectan con el túnel de descarga del aliviadero mediante una cubeta de recepción. El túnel evacua los caudales al río Mundo, aguas abajo de la presa por la margen izquierda.

El esquema hidráulico de los desagües profundos de la presa de Camarillas está conformado por dos desagües de fondo, uno en cada margen de la presa, que atraviesan las laderas del congosto y se maniobran desde cámaras excavadas en el interior de las laderas, y por cuatro tomas de agua, que atraviesan el cuerpo de presa y que se maniobran desde dos casetas adosadas al paramento de aguas abajo de la misma.

El mantenimiento y conservación de los distintos equipos de desagües y tomas presenta una gran dificultad por las siguientes razones:

Figura 3. Fotografía del aliviadero izquierdo.

– El elevado número de equipos existentes.
– El mal estado y lo inapropiado de los accesos, que impide que se pueda realizar con facilidad las operaciones lógicas de mantenimiento.
– La escasez de espacio físico en las distintas cámaras de maniobra.
– La necesidad continua de seguir dando servicio a las demandas del embalse.

Es necesario señalar que el cauce del río, inmediatamente aguas abajo de la presa, ha sufrido (durante la construcción, puesta en carga y explotación del embalse) desprendimientos que han ido reduciendo su capacidad hidráulica y elevando, por tanto, los niveles de agua hasta afectar a las galerías de descarga de los desagües y a la operatividad de los mecanismos de control y regulación de los mismos.

Todo ello ha dado lugar a que ambos desagües de fondo se operen con no pocas dificultades y que por tanto, la capacidad de control de niveles en el embalse no sea la óptima, mermando la seguridad del conjunto de las instalaciones.

Por otra parte, hay que señalar que la fuerte variación de los niveles de embalse, motivada por la restricción de la explotación, la forma en que llegan las aportaciones procedentes del trasvase y la urgencia en dar servicio a las demandas existentes, conlleva que las tomas superiores se queden en seco en multitud de ocasiones, por lo que la presa, en tales circunstancias, carece de suficiente capacidad de toma.

La problemática del embalse de Camarillas, se ha visto acentuada por la revisión hidrológica a la que se han visto sometidos en los últimos años los embalses españoles y que ha incrementado de forma apreciable los caudales y volúmenes de los hidrogramas de cálculo, así como al notable incremento de los criterios de seguridad hidráulica, materializado mediante la aprobación, en marzo de 1996, del REGLAMENTO TÉCNICO SOBRE SEGURIDAD DE PRESAS Y EMBALSES.

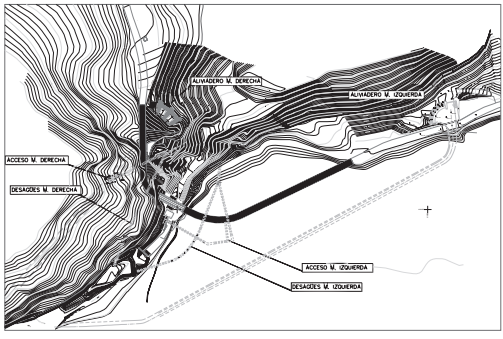

Figura 4. Planta de la presa de Camarillas.

Así, la presa de Camarillas se encuentra clasificada, en función del riesgo potencial que entraña su posible rotura o funcionamiento incorrecto, en la CATEGORÍA A, de acuerdo con lo establecido en la Directriz Básica de Planificación de Protección Civil frente al Riesgo de Inundaciones.

Por ello, se establece una restricción en la explotación del embalse, de tal manera que el Nivel Máximo de Explotación Ordinaria se ubica a la cota 349.79 m, mientras que el N.M.N. es la cota 354.63 m. De esta manera, según el estudio batimétrico del embalse realizado por el CEDEX en 1983 se pierden 13.7 Hm^3 de los 35.837 Hm^3 correspondientes al N.M.N.

Con todo ello, las obras que contempla el Proyecto de Adecuación a la Normativa Vigente del Embalse de Camarillas, han consistido en:

– La rehabilitación de ambos desagües de fondo para contar con unos elementos seguros y funcionales que permitan un adecuado control de los niveles de explotación ordinaria.
– La adecuación de los aliviaderos de superficie, para ampliar la capacidad de evacuación de las avenidas, y recuperar, en la medida de lo posible, la capacidad reguladora del embalse.
– La modernización de las tomas de agua mediante la sustitución de sus distintos elementos hidromecánicos.
– La realización de ciertas obras complementarias como un dique permeable para la retención de elementos flotantes, en la cola del embalse y las actuaciones para la recuperación de la capacidad hidráulica del congosto de los Almadenes, aguas abajo de la presa.

3 GEOLOGÍA Y GEOTECNIA

La cerrada del embalse de Camarillas está situada al comienzo de una estrecha y profunda garganta que ha sido excavada por el río Mundo y que es conocida como “el congosto de los Almadenes”.

Desde el punto de vista geomorfológico, los rasgos más característicos de esta garganta son:

– longitud ≈ 800 m
– profundidad media ≈ 120 m
– anchura media a nivel del cauce ≈ 10 m
– anchura media a nivel de la meseta de coronación ≈ 150 a 200 m

De estos datos se deduce que la pendiente media de los cantiles es de unos 50 a 65 grados; si bien, algo más suave en sus zonas de coronación y algo más fuerte en las proximidades del cauce, donde existen sectores de pendiente sensiblemente vertical que están condicionados por el buzamiento de los planos de discontinuidad del macizo rocoso.

Desde el punto de vista litológico, la mayor parte del congosto de los Almadenes, incluido el propio emplazamiento de la cerrada, se encuentra en el dominio de los materiales calcáreos del Cretácico Superior, Senoniense.

Se trata de una formación constituida, básicamente, por calizas de grano fino, de color beige a gris claro, aspecto masivo y frecuentemente brechificadas.

A escala de detalle, el material presenta indicios de carstificaciones: pequeñas oquedades, diaclasas localmente abiertas y con barniz o relleno arcilloso, etc.

Desde el punto de vista estructural, el rasgo más característico es un amplio pliegue anticlinal de dirección noroeste-sureste, sensiblemente transversal a la dirección del desfiladero, y con una marcada inmersión de su eje hacia el noroeste; es decir hacia el interior del cantil de la margen derecha del congosto.

El dominio del flanco norte del pliegue se extiende a la mitad de aguas arriba de desfiladero, aproximadamente, e incluye el área de ubicación de la presa. En dicho flanco, el rumbo de la estratificación es oblicuo a la dirección media del congosto y su buzamiento varía entre unos 20 y 50 grados hacia aguas arriba; es decir, que presenta una clara componente hacia el exterior del cantil de la margen izquierda.

El dominio del flanco sur del pliegue se extiende a la mitad de aguas abajo del desfiladero, aproximadamente. En dicho flanco, el rumbo de la estratificación es sensiblemente paralela a la dirección media del congosto y su buzamiento, más suave que en el flanco norte, varía entre unos 10 y 30 grados hacia el exterior del cantil de la margen izquierda.

Este esquema general de la orientación de los estratos, se ve frecuentemente modificado por la presencia de fallas de gran continuidad y fuerte buzamiento, que se agrupan en dos direcciones preferentemente de fracturación: una, coincidente con la dirección del eje del pliegue, y, otra, sensiblemente transversal. A esta segunda familia pertenecería la posible falla que habría facilitado el encaje del cauce del río Mundo a través de la meseta calcárea.

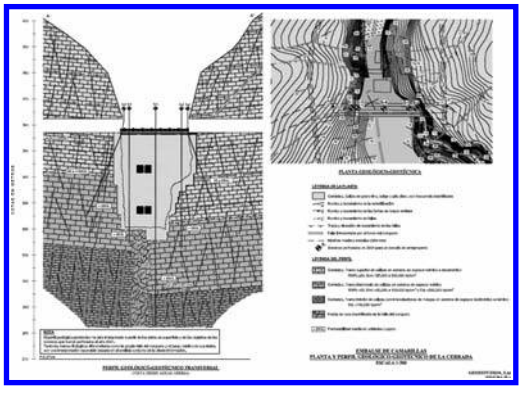

Figura 5. Cortes geológicos en la cerrada.

Se adjunta a continuación un perfil geológico por el eje de la presa.

4 LA REHABILITACIÓN DE LOS DESAGÜES DE FONDO

Las obras proyectadas deben alcanzar los siguientes objetivos:

– Elevar la cota de los dos desagües, de tal manera que su funcionamiento sea compatible con el comportamiento hidráulico del río Mundo, aguas abajo de la presa.
– Contar, en cada desagüe, con dos conducciones independientes, con sus correspondientes mecanismos de control, guarda y regulación en cada una de ellas.
– Es necesario, además, el diseño de unos accesos adecuados a las cámaras de mecanismos de ambos desagües.
– Las obras se deben realizar sin interferir de manera apreciable en la explotación del embalse.

Con todo ello, las obras proyectadas, iguales en ambos desagües, han consistido en lo siguiente:

– Ataguiado de los desagües durante las obras, mediante una de las válvulas BUREAU existentes para ejecución en seco de las obras.
– Colocación, en el tramo de aguas arriba de la galería de descarga actual de cada desagüe de fondo, de una conducción blindada de sección rectangular de 1.50 m de anchura y 2.00 m de altura, soldada con el extremo de aguas abajo de la conducción blindada existente, previa retirada de la compuerta TAINTOR existente y de la compuerta BUREAU de aguas abajo (se conserva la de aguas arriba, que hace las veces

de ataguía) y el picado del hormigón. Por aguas arriba de la ataguía, los dos desagües se dejan en su estado actual. La longitud de esta conducción es, en el desagüe izquierdo de unos 36 m, mientras que en el derecho es de 42.00 m.

– Construcción, en cada desagüe de fondo, de una cámara de carga, en la que evacue los caudales la conducción de 1.50 × 2.00 m y en la que emboquillen, a mayor cota (la impuesta por el funcionamiento hidráulico del río Mundo) dos conducciones gemelas controladas desde la cámara de mecanismos a ubicar inmediatamente aguas abajo. Las cámaras de carga disponen de una conducción de purga de 60 cm de diámetro que tiene salida al exterior, en las cámaras superiores, a cota de coronación de la presa. La altura de las cámaras en cada desagües es de unos 19.00 m, mientras que horizontalmente presentan sección rectangular de 5.00 × 5.475 m. El espesor del revestimiento previsto es de 1.25 m.
– Ejecución de una cámara de mecanismos en cada desagüe, para el control, guarda y regulación de cada una de las conducciones gemelas en las que se ubican dos válvulas BUREAU y una compuerta TAINTOR, que facilita un desagüe bajo compuerta a la galería de descarga. A estas cámaras de mecanismos se accede desde los túneles ubicados en prolongación con la carretera de coronación de la presa, a través de los pozos que contienen, cada uno de ellos, una escalera metálica y un ascensor—montacargas, la conducción de aducción de aire por debajo del filete líquido para el desagüe bajo compuerta y la conducción de purga de las cámaras de carga.

Las cámaras de mecanismos se han proyectado de sección rectangular abovedada superiormente, de unos 7.30 m de anchura interior, 4.90 m de altura en hastiales y 6.40 m de altura en la clave de la bóveda.

El desagüe bajo compuerta se realiza, en cada desagüe, a una galería de sección rectangular abovedada superiormente de 2.00 m de anchura en la base, con 2.00 m de altura en hastiales y 3.00 m de altura en la clave de la bóveda. Estas galerías presentan una disposición rectilínea en planta, con una pendiente del 1%. La longitud de la galería de descarga del desagüe izquierdo es de 34 m mientras que la del derecho es de 46 m.

Los desagües proyectados disponen de una capacidad hidráulica de cálculo de unos 25 m^3/s cada uno.

Finalmente, es necesario señalar que los tramos de las galerías de descarga actuales que se abandonan (desde las cámaras de carga hasta el extremo de aguas abajo) y que discurren por debajo de las nuevas galerías de descarga, se hormigonan.

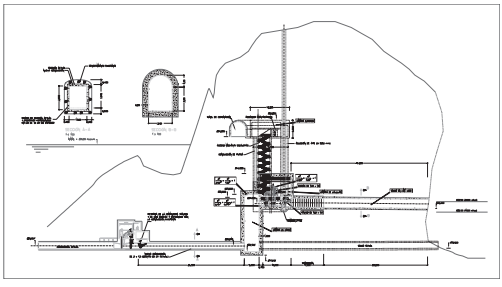

Figura 6. Obras en los desagües de fondo. Perfil longitudinal.

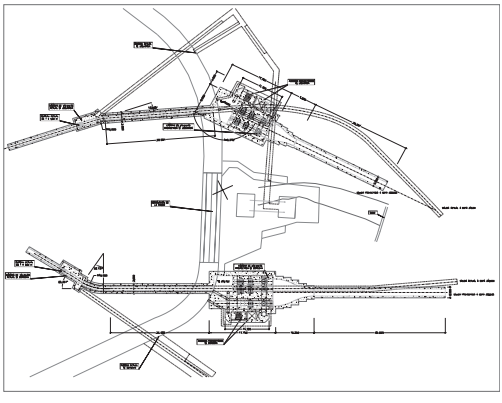

Figura 7. Obras en los desagües de fondo. Planta.

A continuación se presenta una figura con un perfil longitudinal del desagüe derecho. En ella se aprecian las distintas partes descritas: la parte de aguas arriba, hasta la cámara de mecanismos actual, que se deja intacta, la conducción en presión de 1.50 por 2.00 m, que desagua en la cámara de carga, de la que toman agua, a mayor cota, dos conducciones gemelas controladas, cada una por dos compuertas BUREAU y una TAINTOR, siendo esta última la que propicia un desagüe bajo compuerta a la galería de descarga proyectada, que a su vez desagua en el río Mundo.

También se incluye un esquema de la disposición de los dos desagües en planta.

5 ACCESOS A LOS DESAGÜES DE FONDO

Como ya se ha comentado, el acceso proyectado a las cámaras de mecanismos consiste en sendos pozos que alojan en su interior una escalera metálica y el hueco para la instalación de un ascensor—montacargas.

Previamente y para cada pozo, es necesario abrir, desde cada uno de los túneles que acceden a la coronación de la presa, una galería que habilite,

en inicio, una plataforma desde la que ejecutar el pozo y a posteriori, el conjunto de instalaciones comentadas: escalera metálica, ascensor—montacargas (en la parte superior de la galería se dispone la cámara de mecanismos), aducción de aire al desagüe y conducción de purga de la cámara de carga.

La excavación de esta cámara presenta una sección rectangular abovedada superiormente, de 7.75 m de anchura, 3.00 m de altura en hastiales y arco de medio punto.

Se adjunta una planta y una sección vertical de estas cámaras superiores:

Los pozos que conforman el acceso a las cámaras de mecanismos de ambos desagües presentan una sección rectangular de 3.75 m anchura por 11.725 m de longitud y achaflanada en dos de sus esquinas. Como se ha comentado, los pozos alojan en su interior un ascensor—montacargas, una escalera metálica, los conductos de aducción aire de los desagües circulares de 1.25 m de diámetro, que se prolongan verticalmente hasta darles salida al exterior en la ladera, y los conductos de purga de las cámaras de carga, circulares de 60 cm de diámetro. El revestimiento proyectado es de 75 cm. Los pozos tienen una profundidad aproximada de unos 24 m, medida desde la cota de superior (la de coronación de la presa) hasta la cota inferior, coincidente con la inferior de las cámaras de mecanismos.

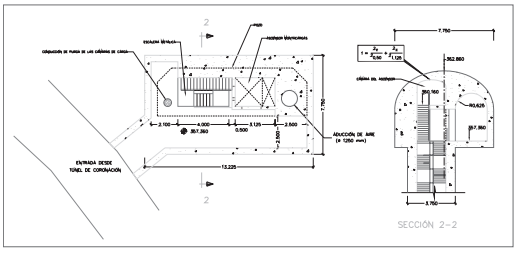

Figura 8. Pozo de acceso a los desagües de fondo.

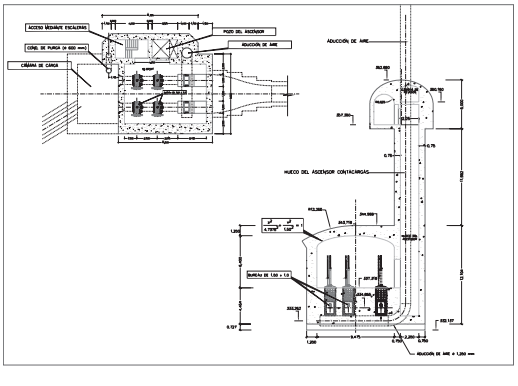

Figura 9. Cámaras de mecanismos de los desagües de fondo.

Se presentan a continuación una planta y una sección transversal de los pozos, en donde se puede apreciar la disposición relativa de los distintos elementos.

6 ADECUACIÓN DE LOS ALIVIADEROS DE SUPERFICIE

Las obras de los aliviaderos se han proyectado bajo las siguientes premisas.

1. La reforma de la embocadura del aliviadero izquierdo es viable, de tal manera que sobre la playa de alimentación del mismo se puede construir un nuevo vertedero de capacidad hidráulica más elevada que la de la embocadura actual.
2. El estudio del túnel de descarga del aliviadero izquierdo muestra que posee una capacidad hidráulica limitada (de 230 m^3/s). La ampliación de esta capacidad hidráulica es difícilmente abordable desde un punto de vista constructivo. Este túnel, presenta un trazado curvo en planta, lo cual desaconseja forzar su capacidad de transporte.
3. Respecto de la adecuación del aliviadero derecho es difícilmente ejecutable la ampliación de la longitud de los vanos. Con ello, la reforma de la embocadura del aliviadero derecho exige rebajar la cota del umbral de apoyo de las compuertas (en la actualidad este umbral se encuentra a la cota 352.00 m). La cimentación de la embocadura actual recomienda no rebajar este umbral en más de 2 m.
4. La cota de coronación de los puentes de paso sobre los dos vanos que conforman el aliviadero derecho es la 356.00 m, inferior a la de coronación de la presa. Si se pretende agotar los resguardos durante el paso de las avenidas de cálculo, es necesario elevar estos puentes.
5. El túnel de descarga del aliviadero derecho presenta una disposición hidráulicamente más adecuada que la correspondiente del aliviadero izquierdo, ya que presenta una rápida tras la embocadura y es rectilíneo en planta. Admite una descarga de hasta 500 m^3/s.

Con todo ello, la solución a desarrollar debe ser la siguiente:

– Máximo aumento de la capacidad hidráulica del aliviadero derecho, mediante el rebaje de la cota del umbral en dos metros. El túnel de descarga existente puede soportar este aumento de la capacidad hidráulica.
– Adecuación de la embocadura del aliviadero izquierdo, de tal manera que se maximice la capacidad de desagüe durante al paso de las

avenidas de cálculo y limitación, en la medida de lo posible, de su uso. Este aliviadero debe considerarse como un aliviadero de seguridad. Así, se ha proyectado un aliviadero de labio fijo, con el umbral algo más elevado que el N.M.N. (se ha dispuesto a la cota 354.60 m), y con tipología de vertedero en laberinto, por presentar este tipo de estructura hidráulica una elevada capacidad hidráulica para sobreelevaciones reducidas de tal modo que agota la capacidad hidráulica del túnel con sobre-elevaciones de lámina reducidas.

Estas reformas permitirían la recuperación casi total de la capacidad de regulación del embalse de Camarillas, al establecer el Nivel Máximo Normal del Embalse a la cota 354.00 m.

6.1 *Aliviadero derecho*

La obra proyectada consiste, por una parte, en la ampliación de la capacidad hidráulica del vertedero mediante el rebaje en dos metros (desde la cota 352.00 m a la 350.00 m) del umbral de apoyo de las compuertas y por otra, la elevación del puente de paso sobre el aliviadero, hasta ubicarlo a la cota 357.50 m, mediante el recrecimiento de la pila—tajamar y de las pilas—estribo de la embocadura.

El conjunto de las obras se ha realizado de tal manera que las compuertas TAINTOR actuales se utilizan en la nueva configuración del aliviadero, de tal manera que no se modifica la posición del eje de las mismas.

Loa obras proyectadas se pueden dividir, a efectos esquemáticos, en la demolición y retirada de la infraestructura actual y en la construcción de la nueva embocadura.

La primera fase de las obras implica la demolición de la caseta de accionamiento de las compuertas TAINTOR, la retirada del puente de paso actual, la demolición del vertedero en ambos vanos del aliviadero, una ligera demolición de la coronación, de los laterales de la pila—tajamar y de las pilas—estribo y la retirada de los accesos a las compuertas TAINTOR. Además, es necesario excavar ligeramente la alimentación a la embocadura, de tal manera que quede conformada por una plataforma a la cota 347.00 m.

La demolición lateral de las pilas se ha proyectado en la zona del escarpe demolido y en un espesor de 25 cm. La demolición del escarpe actual, por su parte, se ha conformado según una superficie escalonada sobre la que apoyar el nuevo.

Se adjunta una figura de las demoliciones.

Previamente a la reconstrucción de la embocadura se realizará en las superficies de demolición un refino de las mismas y se aplicará una resina epoxídica que permita un adecuado pegado de las superficies.

La fábrica nueva, además, se anclará a la antigua mediante una red de bulones.

El perfilado de las pilas, a partir de la ligera demolición de las mismas, se ha proyectado con las mismas formas que las actuales y en prolongación, por tanto, de la parte de las mismas que no han sufrido demolición.

El escarpe proyectado tiene el umbral, como se ha dicho a la cota 350.00 m.

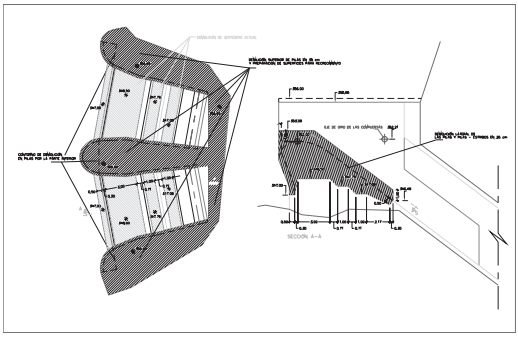

Figura 10. Obras en el aliviadero derecho. Demoliciones.

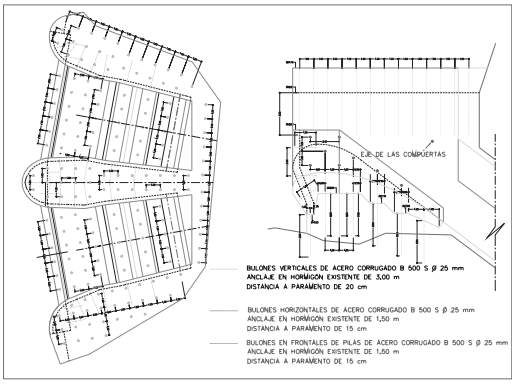

Figura 11. Obras en el aliviadero derecho. Anclajes.

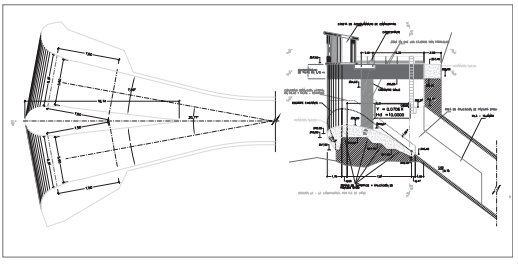

Figura 12. Obras en el aliviadero derecho. Obra terminada.

Los vanos de la embocadura, así proyectada, tienen una anchura de 6.31 m, y resultan convergentes según su disposición inicial con un ángulo de 7.59° hasta enlazar, en el túnel de descarga, adoptando una única sección.

Se adjunta una figura.

El aliviadero proyectado posee una capacidad hidráulica, tal que durante el paso de la avenida de 1.000 años, en situación de pronto, evacua 370 m^3/s, mientras que durante el paso de la avenida de 5000 años evacua hasta 500 m^3/s.

6.2 *Aliviadero izquierdo*

La reforma consiste en la demolición del vertedero actual para la construcción, sobre la playa de alimentación del aliviadero, de un vertedero en laberinto. La selección de esta tipología de vertedero se ha basado en la necesidad de contar con nuevo elemento hidráulico de elevada capacidad de evacuación para sobreelevaciones reducidas de embalse, habida cuenta de que la coronación de la presa se ubica a la cota 357.36 m (la coronación del elemento impermeable se encuentra 40 cm por encima de ésta) y que el resguardo disponible para la evacuación de las avenidas de cálculo es, por tanto, reducido.

No obstante, esta supuesta ventaja—mayor capacidad hidráulica para iguales condiciones de espacio disponible de vertedero—resulta indeterminada cuando la elevación del nivel de embalse sobre el umbral del vertedero crece por encima de determinados valores.

El vertedero en laberinto proyectado se dispone sobre una directriz circular de tal manera que los ejes de los muros longitudinales son tangentes a esta directriz (de 27.00 m de radio y 149.83° de ángulo de desarrollo) y queda delimitado lateralmente por dos muros—cajero laterales que son los dos radio-vectores extremos de la referida directriz circular.

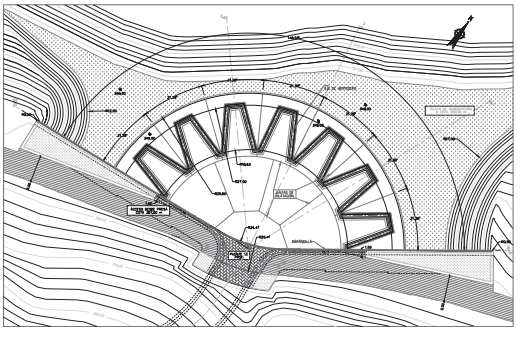

Figura 13. Obras en el aliviadero izquierdo. Obra terminada.

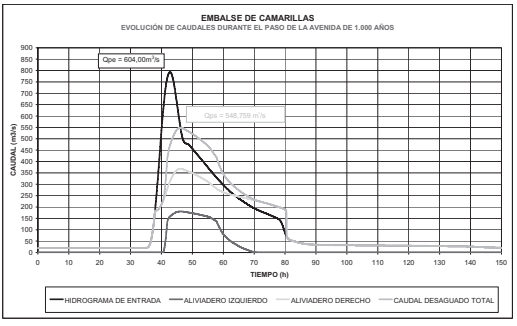

Figura 14. Laminación de la avenida de 1.000 años. Situación de proyecto. Evolución de caudales.

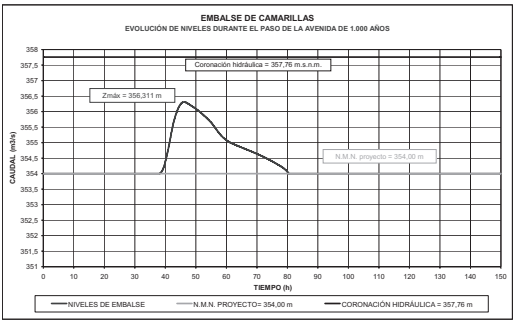

Figura 15. Laminación de la avenida de 1.000 años. Situación de proyecto. Evolución de niveles de embalse.

La reforma queda por tanto conformada por una playa de alimentación que se abre hacia el embalse y que alimenta de agua al laberinto y una cubeta de recepción que converge hacia el túnel de descarga, cuya embocadura se acondiciona a la nueva disposición del elemento hidráulico.

El aliviadero de seguridad proyectado, con el umbral 60 cm por encima del nuevo N.M.N del embalse, sólo entrará en funcionamiento para avenidas superiores a la de 100 años de periodo de retorno y tiene una capacidad hidráulica máxima de 230 m^3/s, de tal manera que se evita que el túnel de descarga entre en carga incluso durante el paso de la avenida de 5000 años de periodo de retorno.

El proyecto contempla otras actuaciones complementarias, tales como la mejora de las condiciones hidráulicas del río Mundo, aguas abajo de la presa, así como la construcción de una obra de retención de flotantes en la entrada del embalse.

Finalmente hay que decir se ha realizado una planificación cuidadosa de cada actuación, al objeto de que se acometa garantizando la seguridad durante su ejecución, a la par que afectando mínimamente a la explotación del embalse.

7 CONCLUSIONES

En el año 2006 se redactó el Proyecto de Adecuación a la Normativa Vigente de los distintos elementos hidráulicos de la presa de Camarillas.

Las obras proyectadas resuelven los indeseables problemas de interacción con el río Mundo, aguas abajo de la presa, y suponen la modernización y adecuación a la Normativa Vigente de ambos desagües de fondo contando con una capacidad, en cada uno de ellos de unos 25 m^3/s. Se proyecta, asimismo, el acceso a las nuevas cámaras de mecanismos mediante sendos pozos en los que se alojan un ascensor—montacargas, una escalera y la aducción de aire a las compuertas de regulación.

Las obras proyectadas en los aliviaderos se ciñen a las embocaduras. En el derecho se consigue un aumento de su capacidad hidráulica, desde unos 240 m^3/s hasta 370 m^3/s de capacidad hidráulica de cálculo (paso de la avenida de 1000 años de periodo de retorno). En el caso del paso de la avenida de 5000 años, el caudal evacuado por el aliviadero derecho es de 500 m^3/s, valor éste que es el límite de la capacidad del túnel de descarga existente funcionando en lámina libre. El aliviadero izquierdo se reconvierte en un aliviadero de seguridad de tipo laberinto, de tal modo que sólo entra en funcionamiento para avenidas elevadas y superiores a la de 100 años de periodo de retorno, y que es capaz de desarrollar toda la capacidad hidráulica necesaria (hasta 230 m^3/s, valor límite del túnel de descarga existente en funcionamiento en lámina libre) con sobre-elevaciones de lámina reducidas.

Se adjunta un gráfico con la evolución caudales durante el paso de la avenida de 1000 años de periodo de retorno, en el que se aprecian los caudales evacuados por cada uno de los aliviaderos.

Asimismo se adjunta otro gráfico, correspondiente al mismo cálculo, pero con la evolución de los niveles de embalse, en el que verifican la idoneidad del resguardo obtenido.

Las obras contempladas permitirían la recuperación casi completa de la capacidad de regulación original de la presa, al situar el Nivel Máximo Normal del embalse a la cota 354.00 m y cumpliendo las todas las disposiciones de la Normativa Vigente en cuanto a seguridad hidráulica.

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Mogoto Dam—case study with regard to maintenance work done on the existing outlet works

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ABSTRACT: This case study describes some of the major problems experienced during the upgrading and the refurbishing of the outlet works at Mogoto dam, as well as lessons learnt during the exercise.

1 BACKGROUND & HISTORY

The Mogoto Dam is a Category 3 earth embankment dam situated in the Limpopo Province, district of Capricorn, on the Mogoto River, approximately 45 km south of Polokwane. The dam is 38 m high and was constructed in 1924. The present owner is Limpopo Department of Agriculture and the dam provides water for irrigation to the nearby Zebediela Estates.

Due to the dam's safety requirements and based on a safety inspection conducted in March 2004, certain remedial works to the existing outlet works had been executed during February/March 2009.

Work could not be finished entirely due to practical problems which have been encountered and had to be addressed by the project team. The lessons learnt from doing work on old dams can be applied to similar dam related work in future. Many service providers and professional persons involved in the dam engineering field may benefit from this case study.

1.1 *Properties of dam*

The dam is registered and classified as a Category 3 dam with a significant hazard potential rating at DWEA (Dam Safety Office). It has a maximum wall height of 38 m with a total non-overspill crest length of 280 m with a gross storage capacity of $2{,}7 \times 10^6$ m^3.

The dam furthermore consists of an open side channel spillway equipped with collapsible crest gates. The core consists of a combination of clay and concrete of which the latter forms the bottom section.

1.2 *History of dam*

The dam was initially constructed and commissioned during 1924. Ingham & Steward was the engineers and Pauling & Company was responsible for the construction. During that time, Zebediela Citrus (Pty) Limited was the owner.

During 1939 the crest was raised after substantial settlement had occurred most probably due to poor construction. During 1976 a relative large sinkhole formed within the left bank. This problem was rectified after substantial grouting and remedial works. The Engineers tasked with the above works were Mr. J.E.B. Jennings (Wits University) and Mr. H.I. Schwartz.

Various control and monitoring instruments were also installed and applied during that time

Figure 1. Location of dam. Applicable 1:50 000 Topo Map—2429 AC.

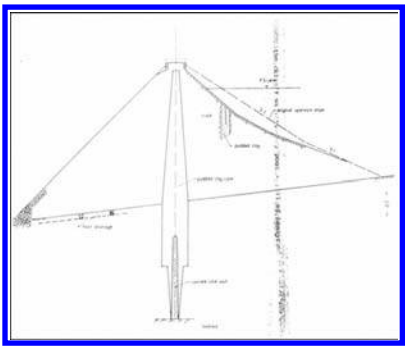

Figure 2. Typical cross-section through embankment.

(i.e. Piezometers for water table readings and Telescopic Bench Marks).

1.3 *Hydrology*

Mogoto dam has a catchment area of 40 km² with an average MAP of 750 mm.

Utilizing the empirical TR137 method at K_e-region 5 the following flood peak values and parameter set were derived;

Q_{20}	(RI of 1 in 20 years)	127 m³/s
Q_{50}	(RI of 1 in 50 years)	242 m³/s
Q_{100}	(RI of 1 in 100 years)	311 m³/s
Q_{200}	(RI of 1 in 200 years)—RDD	387 m³/s
RMF	(Regional Maximum Flood)*	637 m³/s
RMF	(Regional Maximum Flood)**	519 m³/s
SED	(Safety Evaluation Flood)*	796 m³/s
SED	(Safety Evaluation Flood)**	660 m³/s
PMF	(Probable Maximum Flood)	241 m³/s

* *Before attenuation through dam basin*
** *After attenuation*

1.4 *Outlet works*

As can be seen from the photos below, the existing outlet works consist of a brick masonry constructed outlet tunnel equipped with a series of gate valves accessed by a walkway.

During the detail design regarding the proposed betterment works to be done to the outlet works, it was anticipated that the tunnel only extend from the core to the downstream end (Refer to photo 1). This however was far from the real situation. Refer to paragraph 3 in this paper.

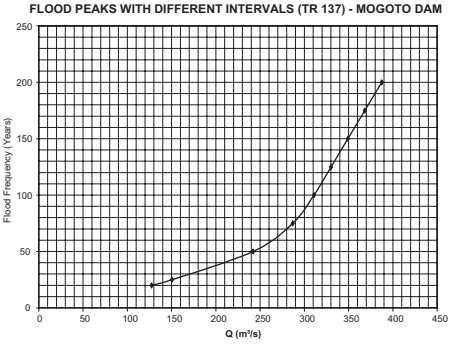

Figure 3. Flood Peaks 1.

2 WHAT HAD LEAD TO THE CASE STUDY?

A compulsory Dam Safety Investigation was required by DWEA as the dam was neglected for a long time and was in a very bad shape in terms of dam safety standards.

The required dam safety inspection was then conducted in March 2004 which had led to the following recommendations which were made;

2.1 *Recommendations made following the dam safety inspection*

- Cut grass and trees on embankment (Refer to photo 3).

Figure 4. Photo 1.

Figure 5. Photo 2 .

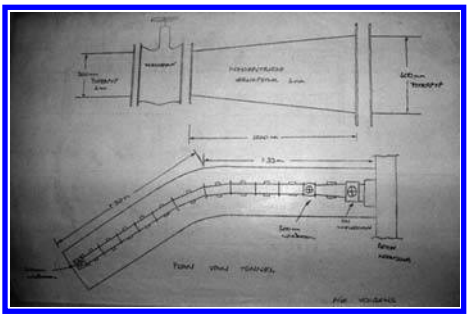

Figure 6. Schematic illustration of outlet / pipe.

- Collapsible gates in spillway to be removed (refer to photo 4).
- Replace / service outlet valves and access walkway (Refer to photos 1 & 2).
- Re-instate and operate monitoring instruments.
- Compilation of a formal operation and maintenance manual with emergency preparedness plan.

Following the recommendations made in the safety inspection report, the Limpopo Department of Agriculture (LDA), instructed PG Consulting Engineers to proceed with the necessary design and documentation for tender purposes in order to upgrade the dam to acceptable engineering standards. Preliminary investigations into the dam had revealed the following;

- Very little "As-Built" information was available.
- No records of quarterly and/or annual maintenance inspections done by Zebediela Estates were available.
- Monitoring procedures and record keeping were stopped since 1976.
- No general maintenance works were done.
- The dam was really neglected and in a bad shape.

Figure 7. Photo 3.

Figure 8. Photo 4.

Funds were allocated by the Limpopo Department of Agriculture (LDA) in June 2008 after which the recommended work has been put out on tender during August 2008. The site was handed over to the Contractor in January 2009.

3 WHAT WAS THE PROBLEM?

3.1 *Initial assumed situation*

As very little "As-Built" information was available for the design of the refurbishment and upgrading of the existing outlet works at the dam, it was anticipated that the outlet tunnel was plugged at the upstream side. The 300 mm-dia steel outlet pipe in the tunnel is linked to a 600 mm-dia gate valve which is situated close to a concrete wall / plugged section. Because of this it was further anticipated that the upstream intake would have a diameter of 600 mm only, which needs to be sealed in order to remove and replace the existing 600 mm faulty valve.

It was therefore specified in the tender to utilize commercial divers to locate the upstream 600 mm-dia inlet and to seal it off in order to proceed with the recommended betterment works. After the tender process, commercial divers were hired during construction to assess the situation while the water level in the dam was at a depth of approximately 25 m. As known, diving at this depth is quite expensive.

The Zebediela Estates was also not prepared to empty the dam as they need the water for irrigation of citrus.

3.2 *Actual situation*

Following the actual diving investigation, the assessment made by the divers had revealed a total different scenario as described below;

The opening expected to be 600 mm in diameter were in actual fact 3 m × 3 m in size. Refer to illustration below. The plugged end of the tunnel, initially anticipated to be at the upstream side was found to be closer to the centre of the embankment. The tunnel extends upstream for approximately

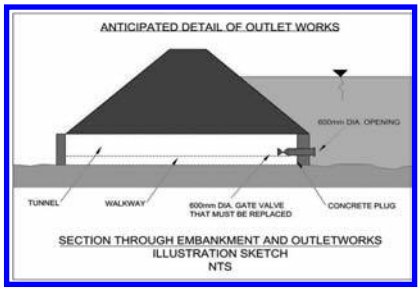

Figure 9. Anticipated detail of outlet works.

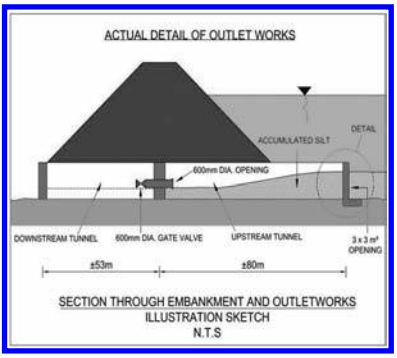

Figure 10. Actual detail of outlet works.

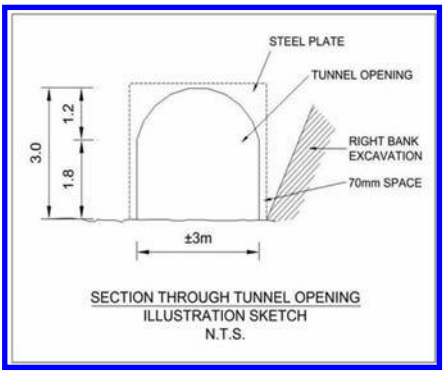

Figure 11. Section through tunnel 1.

another 80 meters where the opening was located, covered by a fixed steel grid. A huge volume of silt deposit was also found at the opening as well as inside the tunnel.

It was decided to remove the grid in front of opening by cutting under water. During this exercise some of the silt deposit also had to be removed. This changed the scope of work. The budget was also limited and allowance was only made for approximately one (1) week of diving. The situation was then re-evaluated by the Engineer, Dive Team, Contractor and Client—*back to the drawing board.*

3.3 *Adapted solution*

A new solution was recommended and adapted in order to close off the larger opening while limiting the diving time. A steel plate had to be designed and manufactured on site to be placed in front of the opening (Refer to illustration below). A motivation was draft and submitted to LDA (owner) in order to obtain additional funding. This again also caused a delay in terms of contract time as the new Verification Order (VO) had to be approved.

After approval was received, the steel plate was designed, manufactured and put into place, but however, as they say; "*the Proof of the pudding lies in the EATING*!" and the steel plate somehow deformed during installation.

The following probable causes were identified for the deforming of the steel plate / unsuccessful closing off of the opening;

- Limited / insufficient support when the steel plate was placed into position.
- Very poor to no visibility under water, making it very difficult for the divers to see.
- At this phase of the exercise, the divers were already exhausted.
- Very little progress was made for some time, which causes,
- the morale of divers to reach a low point.

Because of the above, the project was temporary halted. It was then decided to adopt an alternative strategy by waiting till the dam is empty. From the start however, this option was not really acceptable, as stored water is a scarce commodity.

4 LESSONS LEARNT FROM THE CASE STUDY

The following summary gives an aftermath of the lessons learnt during this case study;

- Size does count—Need we say more?
- "As-Built" information is vital to decision-making during preliminary design.
- Doing maintenance work on old dams must be handled with *caution*.
- Before going out on tender, if possible, arrange for a recognizance inspection by divers to assess the situation as part of the detail design.
- **NB!** Avoid making assumptions.
- If possible, arrange for upgrading works to be done when the water level in the dam is low. Especially when it involves refurbishment or replacing of outlet works.

ACKNOWLEDGEMENTS

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Reparación de los desagües de fondo de la presa de la Fuensanta, utilizando las válvulas de seguridad como escudos frontales

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RESUMEN: La presa de la Fuensanta se construyó hace más de 75 años. Los desagües de fondo, alojados en el tapón de cierre del túnel de desvío del río, son dos conductos rectangulares de sección abocinada, maniobrados con válvulas de compuerta que evacuan directamente el vertido a la galería. Dada la antigüedad de los equipos mecánicos, y su estado de deterioro, se ha procedido a su renovación. Los problemas habituales de estas actuaciones (existencia de sedimentos y desprendimientos en el área de la embocadura, falta de conocimiento de las formas geométricas de las zonas ocultas por el agua, defectos de construcción no documentados, deterioro de los blindajes, dificultad de acceso, etc) se han resuelto en esta obra acondicionando las antiguas válvulas de seguridad de los conductos para utilizarlas como escudos frontales, lo que ha permitido sustituir y ampliar los blindajes de acero, e instalar las nuevas compuertas de aguas abajo.

ABSTRACT: Fuensanta dam was constructed 75 years ago. The bottom outlets were located on the diversion tunnel. These consist of two rectangular conduits regulated by two gate valves (for each one) which are situated on the closure plug. Water released by the outlets returns to the river through the diversion gallery in free surface flow. Due to the age of the mechanical equipment and its grade of deterioration, it has been proceed to its renovation. Common problems and uncertainties of this type of works (existence of silting near the mouth of the outlets, possible instability and failure of rock blocks, unknown of the real geometry underwater, corrosion of the steel components, difficult accesses, etc) have been solved adapting the upstream existing valves as cofferdams, permitting the substitution of the old steel lining and downstream valves and the installation of new radial gates.

1 INTRODUCCIÓN

1.1 *Datos característicos de la presa*

Las obras de la presa de la Fuensanta comenzaron en el año 1929 y se dieron por concluidas en 1933. Está situada en el río Segura, en la zona de cabecera de la cuenca (TM de Yeste / Albacete), tiene una capacidad de embalse de 235 hm^3, e inunda una superficie de 890 ha.

La presa es de gravedad de planta curva, de 82 m de altura, 232 m de longitud de coronación, taludes aguas arriba 0,05/1 y aguas abajo 0,81/1. Dispone de 4 niveles de galerías, conectados entre sí con pozos verticales.

1.2 *Descripción de los desagües*

En la fecha de su construcción, coetánea con la creación de la Confederación Hidrográfica del Segura (año 1926), el diseño del sistema hidráulico (aliviaderos y desagües) se apartaba claramente de la estructura de la presa, disponiéndolo generalmente sobre túneles horadados en la montaña (como si se tratase de presas de materiales sueltos) en lugar de albergarlos en el propio dique de hormigón como es habitual actualmente. El cambio en la concepción del diseño se produjo en España durante la década de 1940.

Los aliviaderos y desagües de la Fuensanta se hallan sobre túneles excavados en la formación

caliza de margen izquierda de la cerrada. Los desagües de fondo discurren por el túnel construido para el desvío del río y posteriormente reutilizado para este fin. Los conductos están moldeados en el hormigón del tapón de cierre, y disponen de un corto tramo con blindaje metálico sobre el que se sitúan las compuertas de maniobra. La cámara de válvulas queda englobada en el tapón, el cual tiene más de 50 m de desarrollo, y en el mismo los conductos moldeados de los desagües cambian gradualmente de sección, tanto en el tramo de entrada como en el de salida, para alcanzar en el tramo blindado (el que aloja a las válvulas de compuerta) una sección de 1,40 $\times$ 2,20 m.

Cuando de construyó la presa se dispuso una doble vía de acceso a la cámara de válvulas: por un lado desde aguas abajo a nivel del pie de presa, a través de un túnel que discurre en paralelo al de los desagües; y por otro desde aguas arriba, a nivel de coronación, a través de un montacargas instalado en un pozo abierto en la ladera izquierda del embalse (este acceso, aunque se conserva como chimenea de ventilación, está hoy fuera de servicio). El montacargas está alojado en el interior de un edificio singular, con un bello diseño arquitectónico.

En cada uno de los conductos de desagüe se instalaron dos válvulas de compuerta rectangulares, con accionamiento oleohidráulico mediante cilindros de baja presión.

1.3 *Necesidad de sustituir las válvulas*

Los desagües de fondo han funcionado satisfactoriamente desde la puesta en carga de la presa, en 1933, con las normales labores de mantenimiento. Sin embargo la antigüedad de las mismas y su larga vida, han acabado produciendo holguras por desgaste natural de las zonas de fricción de los elementos móviles. Ello, unido a las deficiencias propias de un diseño muy anticuado, ha culminado dificultando seriamente las maniobras, lo que ha determinado la necesidad de proceder a su sustitución inmediata y a la rehabilitación integral de los desagües. Estas actuaciones, una vez tomada la decisión, deben ser acometidas con urgencia ya que se crea incertidumbre en el programa de mantenimiento, puesto que se tiene un riesgo latente de que se produzca una avería durante la maniobra que por acodalamiento impida cerrar el desagüe y se vacíe el embalse (lo cual ocasionaría daños muy graves, tanto por la pérdida del agua como por los efectos medioambientales negativos que se generarían), o bien si se inhibe el explotador y no realiza ninguna maniobra con ellas se inicia un proceso de reacomodación del cono de sedimentos que puede llegar a taponar la boca de entrada del túnel, inutilizando los desagües.

2 ACTUACIONES PREVIAS

2.1 *Planteamiento general*

La solución habitual para rehabilitación integral de los desagües de fondo precisa taponar la entrada con un escudo de cierre frontal, dejando el conducto en seco y fuera de servicio mientras se realizan las obras de reparación. Hoy en día, en todas las presas de nueva construcción se dejan instaladas las guías (en acero inoxidable) que permiten insertar desde coronación el escudo de sellado del conducto. Sin embargo esta previsión no se tenía en las de tanta antigüedad como es la presa de la Fuensanta. Hay otras posteriores en las que se dejaron guiaderas de acero ordinario y hoy están inutilizadas por la herrumbre, y otras en donde sólo se dispusieron ranuras locales de acoplamiento no accesibles desde fuera del agua.

En todas estas presas, que no disponen de guiaderas apropiadas para insertar el escudo, éste debe acoplarse con trabajos subacuáticos. En las presas antiguas, sin ranuras de acoplamiento del escudo, se suele preparar un bastidor de enlace que se fija a la embocadura del conducto y una vez que está éste colocado, se baja y acopla el escudo al bastidor. El cierre de las válvulas de equilibrado instaladas en el escudo sella por presión el contacto con el bastidor. El escudo dispone de cámaras de aire para equilibrar el peso sumergido y facilitar su manipulación.

Todo ello ha de realizarse bajo el agua. La operación parece sencilla, pero suele estar plagada de incidentes.

2.2 *Problemas e incidentes habituales*

En los trabajos subacuáticos desarrollados en presas antiguas para colocación de los escudos de sellado es frecuente que aparezca algún contratiempo que obligue a realizar actuaciones adicionales que, cuando menos, retrasen el plazo de ejecución. Entre los problemas que pueden encontrarse figuran los siguientes:

- Existencia de sedimentos en el campo de trabajo, que enturbian el agua y reducen la visibilidad. Pueden obligar a realizar una limpieza previa, aspirando el lodo del lecho.
- Existencia de piedras, áreas de desprendimientos, troncos de árboles, tablones, chatarra de obra, etc. Es necesario apartarla, o extraerla, y a veces previamente a trocearla.
- Existencia de irregularidades en las paredes de la embocadura del conducto, que dificultan la colocación y sellado del bastidor. Obligan a realizar trabajos de bucería importantes, demoliendo o adicionando material en la zona de encaje para conseguir una superficie suficientemente

plana (a veces la demolición de protuberancias del hormigón ha producido sorpresas, ya que taponaban coqueras, o vías de agua, que han reaparecido al eliminarlas).
– Las formas geométricas de la embocadura no coinciden con la información que se tiene sobre ella, proveniente del proyecto de construcción. Es bastante frecuente en las presas antiguas que las modificaciones de obra (especialmente las realizadas en las zonas ocultas o difícilmente accesibles) no estén correctamente documentadas.

2.3 *Reconocimiento subacuático*

El sellado con escudos frontales requiere realizar, previamente a la ejecución de las obras, una inspección subacuática en la que obtenga información suficiente para solventar la problemática específica de cada caso, o bien determinar la inviabilidad del proceso. Respecto a ello es importante, en las presas con desagües profundos próximos al nivel del lecho del cauce (y en general en las que existe una potencia de sedimentos en el embalse que ha rebasado el umbral de los desagües), evaluar la peligrosidad (proximidad y altura) del cono de lodos que puede haberse formado en el contorno de la embocadura, ya que éste se halla siempre en un equilibrio precario y existe el riesgo de reptación del material mientras se realizan los trabajos subacuáticos.

3 SOLUCIÓN ADOPTADA EN LA FUENSANTA

3.1 *Estado de la embocadura*

Durante los casi 80 años de explotación de este embalse se ha acumulado una gran cantidad de sedimento, fundamentalmente en el entorno próximo a la presa (se trata de un embalse hiperanual, en una cuenca de recursos escasos y riadas importantes, que permanece semivacío muchos años), estimándose que desde el inicio ha perdido unos 25 hm^3 de capacidad.

En la inspección subacuática realizada al inicio de la obra se detectó que se había producido una reacomodación del cono de sedimentos existente en la zona de influencia contigua a la entrada del túnel, que había alcanzado la embocadura. Según la evaluación realizada por el equipo de buzos el espesor de lodos en la boquilla del túnel era de 1,80 m, lo que dificultaba el acceso a la misma. Este espesor de sedimento se reduce gradualmente hacia el interior de la galería. Al pasar sobre el lodo, se remueve y enturbia el agua, por lo que el trabajo se llevó a cabo con visibilidad muy reducida.

Los hallazgos de la inspección fueron desalentadores, ya que el inicio del tapón estaba más lejos de la boca del túnel que lo que figuraba en los planos antiguos de la presa, y las formas geométricas del hormigón de la embocadura estaban adaptadas a la sección irregular en que había quedado la excavación del túnel, y la entrada disponía además de un tajamar central que dificulta la posible colocación de un escudo frontal. Se comprobó que la transición para alcanzar la sección rectangular del conducto tenía un gran desarrollo, con abocinamiento gradual y formas geométricas no convencionales, lo que impedía el acoplamiento del bastidor. Como consecuencia de este conjunto de vicisitudes, y a la vista de la peligrosidad que entrañaba la gran acumulación de sedimentos existente en el área de entrada al túnel, se desestimó la solución de utilizar escudos frontales sobrepuestos en la boquilla de los conductos, optándose por la solución alternativa consistente en utilizar para esta función a las compuertas de aguas arriba de los desagües, acondicionándolas previamente para este cometido.

3.2 *Habilitación de las compuertas antiguas para su empleo como escudos frontales de cierre*

La solución de habilitar las compuertas antiguas de aguas arriba para su empleo como escudos frontales de cierre, puede ser la opción más favorable cuando confluyen algunas de las circunstancias siguientes:

– Existe la posibilidad de prescindir de esta válvula durante la explotación ordinaria, sin menoscabo de la seguridad, para lo cual el desagüe ha de disponer de otros dos elementos de cierre operativos, o bien cabe la posibilidad de instalar un tercero por aguas abajo.
– Existe riesgo para el personal que ha de realizar las operaciones subacuáticas, bien por reptación de los lodos acumulados en el área contigua a la zona de trabajo, o por dificultad del acceso, o por cualquier otro motivo que implique un incremento de peligrosidad en la ejecución de los trabajos de los buzos.
– Existen grandes irregularidades, o deficiencias en las formas geométricas de la embocadura del desagüe, que impiden la colocación del bastidor del escudo.

La necesidad de disponer de un doble cierre en los desagües es obligatorio, por lo que no es viable renunciar a la válvula de aguas arriba si no hay otra que la supla eficientemente. Por lo tanto el cumplimiento del primero de los puntos indicados anteriormente es insoslayable. Sin embargo, los otros dos pueden sólo implicar un mayor encarecimiento de los trabajos subacuáticos, ya que para colocar el escudo en la embocadura del desagüe habrá que tomar medidas adicionales que obvien el problema planteado.

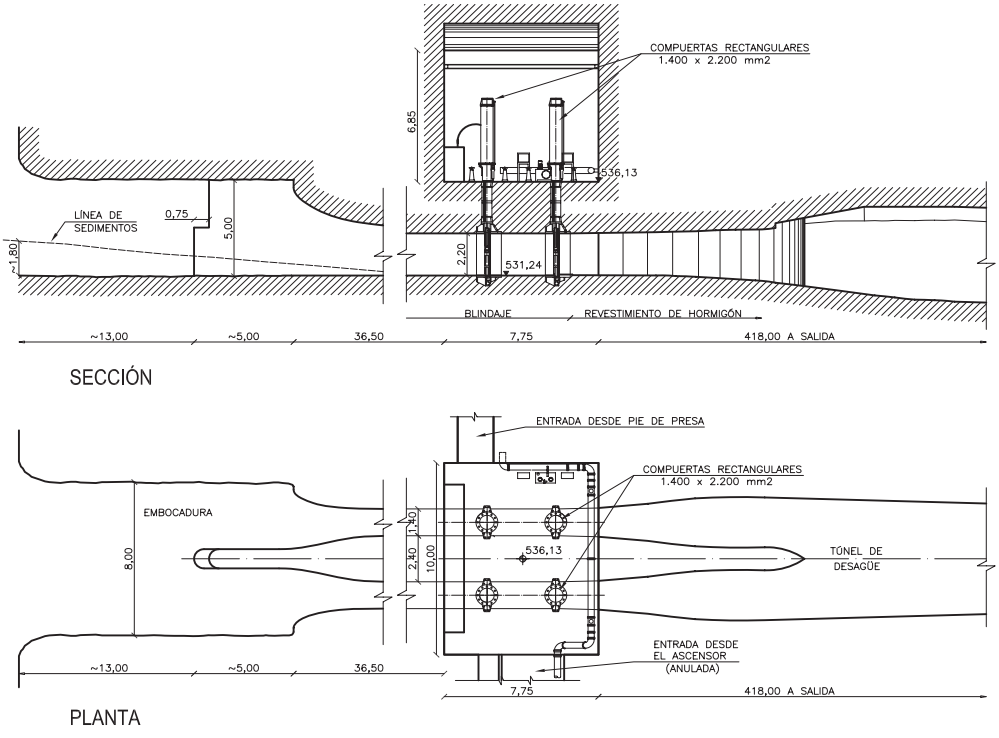

Figura 1. Conductos de los desagües de fondo, antes de la actuación de reparación.

En la presa de la Fuensanta todas las circunstancias aconsejaban desechar la solución de taponamiento de la embocadura con un escudo, por lo que se procedió a la habilitación de la válvula de aguas arriba para realizar esta misión, relegándola de la explotación ordinaria, ya que en cada desagüe se dispondrá de una compuerta tipo Bureau (como cierre de seguridad, al sustituir la segunda válvula existente en la cámara) y adicionalmente una compuerta tipo Taintor (como elemento de regulación, instalada en el terminal del conducto en el punto de vertido al túnel).

La habilitación de las válvulas de compuerta antiguas para su nueva función como escudo de cierre es bastante sencilla, ya que se trata de limitar sus funciones, puesto que sólo han de intervenir cuando se quiera dejar en seco el conducto para revisión o reparación de las compuertas de maniobra de aguas abajo. Se han realizado las siguientes operaciones:

– Revisión, limpieza y ajuste de las guías del tablero de la válvula. El equipo de buzos ha procedido también a sellar la entrada del by-pass, para eliminar las fugas de agua que se producen por dicho conducto, ya que la válvula de control de éste estaba averiada. Si la válvula de cierre del by-pass estuviese operativa, es útil mantener en servicio este dispositivo.
– Instalación de una pieza mecánica que permite inmovilizar el vástago del tablero en la posición de apertura total de la compuerta, dejándola transitoriamente fuera de servicio. En caso de que se necesite reutilizarla de nuevo como escudo, previamente debe retirarse esta pieza de fijación, quedando de nuevo en condiciones de operatividad.

3.3 *Ampliación del blindaje de los conductos*

Una vez comprobado el funcionamiento correcto de la válvula de aguas arriba en su nueva función como escudo frontal, se ha procedido a la ejecución de la reparación y ampliación de los blindajes de los conductos, a la sustitución de la segunda válvula de compuerta por otra tipo Bureau, y a la instalación de una nueva compuerta Taintor en el terminal del blindaje, con vertido libre al túnel.

Con la ampliación del blindaje de los conductos hacia aguas abajo se ha corregido el problema latente, con riesgo de cavitación, que existía en el tramo terminal de vertido al túnel. Los conductos, al término de la válvula de compuerta de aguas abajo, tienen una sección divergente, a modo de transición, con la que pasan de la forma rectangular del desagüe a la del túnel. Esta transición es similar a la de la embocadura, pero colocada

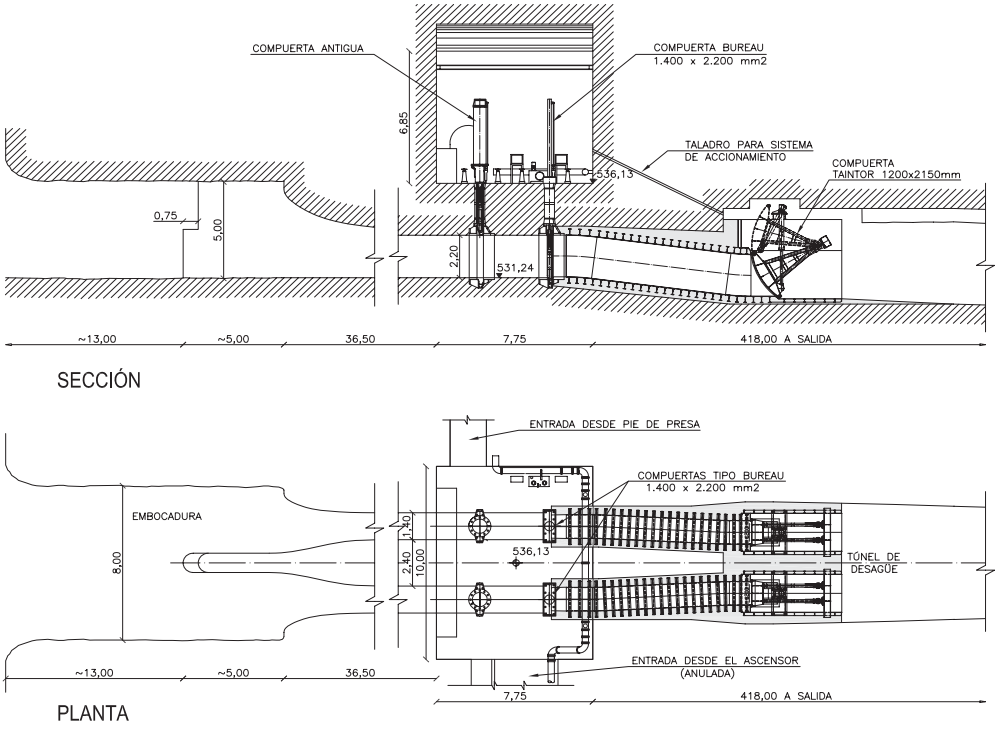

Figura 2. Conductos de los desagües de fondo, al término de la actuación.

en sentido inverso. La circulación del agua a alta velocidad produce depresiones en el contacto de la vena líquida con la pared, con cavitación, por lo que el funcionamiento de este tramo de evacuación del caudal siempre ha sido defectuoso y ha producido el deterioro de las paredes del hormigón.

Se ha modificado por ello el diseño de todo este tramo terminal del conducto (entre la compuerta Bureau y la Taintor) con dos actuaciones superpuestas:

– Regularización de la sección, para mantener constante el área y las formas del conducto.
– Blindaje con chapa de acero de todo el tramo, dejando una superficie perfectamente lisa.

La regularización de la sección ha exigido demoler parcialmente los hormigones de la transición para ganar espacio y, una vez colocados los blindajes de acero, se han rellenado los huecos con hormigón bombeado y posteriormente se han inyectado con lechada de cemento las zonas poco accesibles para sellar los huecos en los que no se puede asegurar que el hormigón haya penetrado correctamente a todo el recinto que queda tras el blindaje, como es la zona plana horizontal existente bajo la solera de los conductos o la clave de la sección del túnel.

3.4 *Instalación de las nuevas compuertas*

La sustitución de la compuerta de aguas abajo por otra de nuevo diseño tipo Bureau, dotada de un cilindro de alta presión, se ha realizado con la siguiente secuencia de actividades:

– Desmontaje del cilindro, campana y tablero de la compuerta, y del by-pass y aducción de aire de ésta.
– Montaje del nuevo equipamiento: tablero tipo Bureau, campana, cilindro hidráulico de alta presión, by-pass y aducción de aire.
– Sustitución de la central oleohidráulica antigua por otras nuevas individualizadas para cada conducto, con doble grupo de presión, cuadros de mando y eléctrico.
– Reposición de toda la instalación hidráulica.

La instalación de las nuevas compuertas Taintor, ubicadas en el punto terminal del blindaje de los conductos, es una operación sin complicaciones, ya que se dispone de espacio en la amplia sección del túnel (6 m de ancho por 5 m de altura), por el que se puede acceder con la maquinaria y medios auxiliares de montaje a través de éste. Con estas compuertas se regulan los caudales evacuados por los desagües de fondo, que circulan por el túnel en lámina libre.

Las actuaciones realizadas han sido las siguientes:

- Demolición de los hormigones de la transición de salida, del tajamar central y del techo de la galería, con objeto de disponer del espacio suficiente para instalar los blindajes de acero de los conductos y las compuertas Taintor situadas en el terminal de éstos.
- Instalación de los blindajes de los conductos y de la zona de deslizamiento de las compuertas, y hormigonado del trasdós.
- Montaje de las compuertas Taintor.
- Perforación en el hormigón del revestimiento para abrir el orificio de paso del equipamiento de maniobra de las compuertas Taintor hasta la cámara de válvulas, conexionándolas a la central oleohidráulica correspondiente (con la misma con que se mueven también las compuertas Bureau instaladas en la cámara).

3.5 *Remozamiento de espacios*

Las obras de rehabilitación integral de los equipos mecánicos de los desagües deben aprovecharse para remozar también los espacios en que se ubican. Este criterio se ha aplicado en la presa de la Fuensanta, en donde se ha procedido a la reparación del edificio del montacargas, en el que se halla el antiguo pozo de acceso a la cámara de válvulas, hoy no utilizable aunque funciona como respiradero, y ante una situación de emergencia se podría llegar a la cámara a través de éste.

También se ha procedido a impermeabilizar las paredes de la cámara de válvulas y de la galería de acceso, disponiendo una capa nueva de recubrimiento que desvía las filtraciones hacia los canales de drenaje.

4 CONCLUSIONES

En las presas antiguas, en las que se precisa reparar integralmente los conductos de los desagües de fondo y no disponen de ningún dispositivo para colocar un escudo frontal de cierre, puede ser una buena solución habilitar la válvula de aguas arriba para este fin, evitando los trabajos subacuáticos que en otro caso se precisarían llevar a cabo.

La disposición de los escudos frontales para dejar en seco los conductos exigen colocar previamente un bastidor de acomplamiento, y al término de los trabajos se precisa retirar todo este equipamiento. Los problemas habituales con que se encuentran los buzos cuando se realizan estas actuaciones se deben a la presencia de sedimentos, mala visibilidad, existencia de obstáculos en el entorno de la embocadura, irregularidades de acabado en la zona de acoplamiento del bastidor y, a veces, unas formas geométricas distintas de las esperadas.

El empleo de la válvula de aguas arriba evita la necesidad de colocar el escudo con buzos, obviando la problemática antes referida, y tiene además la ventaja de poder utilizarla, con esta finalidad, siempre que se precise a lo largo de la explotación de la presa. Pero para que pueda cumplir este cometido se necesita que el desagüe disponga de otros dos elementos de cierre, de manera que se pueda renunciar durante la explotación ordinaria al uso de la válvula de aguas arriba, la cual permanecería siempre abierta.

En consecuencia se recomienda que en los desagües de fondo de las presas de nuevo diseño se disponga siempre de tres elementos de cierre: el primero con la función de escudo, el segundo sería el cierre de seguridad y el tercero sería el cierre de regulación.

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Sustitución de compuertas en órganos de desagüe de presas de la Confederación Hidrográfica del Duero

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ABSTRACT: Confederación Hidrográfica del Duero is the owner of 18 reservoirs, with a total capacity of 2.285 hm^3 (3yd^3). The oldest reservoir is Cervera, which was finished in 1923. Since many years ago, the sluice gates of this dams has been replaced, as their design, materials and requirements become obsolete, besides, it has sense as they are getting damaged because of their use and because of the climate conditions of the places where they are located. Until nowadays, it has been done or put out to contract 14 dams. The first work started in 1956. We take in consideration only the works that means a total replace of the sluice gate, not the partial restorations, replacement of the system, electrical system and so on.

1 INTRODUCCIÓN

La Confederación Hidrográfica del Duero es titular de 18 embalses de regulación, con una capacidad total de 2.285 hm^3. El embalse más antiguo es del Cervera, terminado en 1923. El más reciente en explotación es el de Riolobos, terminado en 1998. Además está terminada la presa de Irueña, cuyo embalse será de 110 hm^3, cuya puesta en carga se iniciará próximamente, y está en construcción el de Castrovido, cuyo embalse será de 44 hm^3.

De estos embalses, el mayor es el de Riaño, con 664 hm^3 de capacidad. El más pequeño es el de Pontón Alto, con 7,4 hm^3 de capacidad.

Los primeros elementos colocados en los órganos de desagüe de las presas se construyeron en la segunda decena del siglo XX, con unas características de diseño, materiales y constructivos muy alejados de lo que ahora es habitual, y construidos por empresas que forzosamente no podían tener mucha experiencia.

2 RECOPILACIÓN DE INFOMACIÓN

Se ha realizado una recopilación exhaustiva de todas las actuaciones en las que se ha realizado la sustitución completa del elemento. No se mencionan la reparación o sustitución de la instalación eléctrica, del sistema de accionamiento conservando la compuerta, u otras reparaciones mayores.

La información procede del Archivo General de la Confederación Hidrográfica del Duero.

Para cada actuación se ha obtenido su coste actualizado a 2010. Para ello se ha aplicado el Índice de Precios al Consumo de enero a enero de cada año disponible en la web del Instituto Nacional de Estadística.

En la tabla que figura al final de la comunicación se resumen todas las actuaciones, incluidas la adjudicadas y no iniciadas, y una iniciada, que suman 40,5 millones de euros sin actualizar y 47,7 millones de euros actualizados. La diferencia tan baja indica que se concentran en los últimos años.

Los importes son los correspondientes a la liquidación de la obra.

La fecha que figura es la aproximada de terminación de la obra.

3 PRESA DE CAMPORREDONDO

3.1 *Desagüe de fondo de la margen izquierda*

Se dedica mayor extensión a la primera actuación realizada en una presa de la Confederación Hidrográfica del Duero, en la presa de Camporredondo.

El proyecto de la presa es de 1914. Su presupuesto era de 5.200.000 pesetas. Se iniciaron las obras en 1914, y se terminaron en 1930. Las obras se ejecutaron por Administración. La presa es de gravedad, de planta curva de 220 metros de radio. Taludes vertical y 0,80, altura sobre cimientos de 75,50 m, altura sobre cauce 67,50 m, cota de coronación 1.292,20, longitud de coronación 160 m, y volumen de fábrica 172.400 m^3. El volumen de embalse es de 70 hm^3.

Tenía tres desagües de fondo, dos idénticos en cuerpo de presa, el tercero situado en la galería de desvío. Las compuertas eran idénticas en los tres, con unas dimensiones de 1.000×1.500 mm^2, doble válvula en cada conducto, accionadas mediante

émbolos hidráulicos, que utilizaban como fluido agua a 200 atmósferas de presión.

El proyecto de los desagües es de 1922, se adjudicó en 1923 a "Talleres de Palencia" en 220.000 pesetas, solamente el suministro y montaje, no la obra civil.

La obra se recibió provisionalmente en 1930, y definitivamente en 1931.

Los cuerpos de las válvulas en principio eran de fundición. Se rompieron en las pruebas de taller, y fueron sustituidas por otras de acero dulce. No tenían aducción de aire.

Hasta la puesta en funcionamiento de la central de pie de presa la única forma de regular el embalse eran los desagües de fondo. Solamente había una toma intermedia con un solo conducto de 800 mm de diámetro cerrado por una sola llave de compuerta, y una válvula de nuez también de 800 mm de diámetro instalada en 1942 en una de las tuberías forzadas de la central.

El funcionamiento de las compuertas en aperturas parciales, sin aducción de aire, provocaron daños en los elementos metálicos y en el hormigón del entorno y los conductos.

Aunque se realizaron numerosas reparaciones, las pérdidas del desagüe situado en la galería de desvío llegaron a ser de 200 l/s.

La situación de deterioro era tal que en un vaciado del embalse en 1942 se colocó un tapón en este conducto, por lo que a partir de entonces la regulación se hizo con la central y con la muy limitada de la válvula de nuez.

En el año 1947 se redactó el proyecto de sustitución de las compuertas situadas en la galería de la ladera, en el que se consideró que no podía abordarse la obra en los tres conductos de desagüe de fondo en el mismo contrato, dejando para un proyecto posterior la sustitución de las de los otros dos conductos.

Figura 1. Camporredondo. Compuerta Bureau instalada en 1956.

El presupuesto de las obras era 440.800 pesetas. Se terminaron en 1956, incluyendo en las mismas unos tapones de hormigón en los otros dos conductos de los desagües de fondo.

Las compuertas eran de maquinista y Fundiciones del Ebro, empresa muy conocida en este tipo de instalaciones. Tenemos unas válvulas prácticamente idénticas en Requejada, y las hemos tenido en Arlanzón.

Posteriormente en 2001 se han sustituido los mecanismos de los otros dos desagües de fondo.

3.2 *Aliviadero de margen derecha*

En el aliviadero de la margen derecha había una compuerta de sector flotante, de hormigón. Tenía entradas de agua apreciables, por lo que no podía garantizarse su flotabilidad. Además tenía el eje roto. En 1974 se dejó empotrada en su alojamiento, colocando encima una compuerta Taintor de 12,00 × 4,80 m^2 con accionamiento oleohidráulico.

3.3 *Desagües de fondo y toma intermedia*

En 2001 se terminó la rehabilitación los dos desagües de fondo existentes en el cuerpo de presa, colocándose nuevos blindajes, embocaduras y dos válvulas Bureau en cada conducto de 800 × 1.200 mm^2 de sección. Se ha mencionado anteriormente que los anteriores eran de 1.000 × 1.500 mm^2.

También se rehabilitó la toma intermedia, ampliando el conducto existente, que era de 800 mm de diámetro, pasando a sección rectangular de 1.000 × 1.500 mm^2 de sección, colocándose también nuevos blindajes, una válvula de Bureau de seguridad de 1.000 × 1.500 mm^2 de sección y una Howell-Bunger de 1.500 mm de diámetro de regulación, con concentrador.

Estos trabajos se realizaron vaciando el embalse, que fue posible por disponer inmediatamente

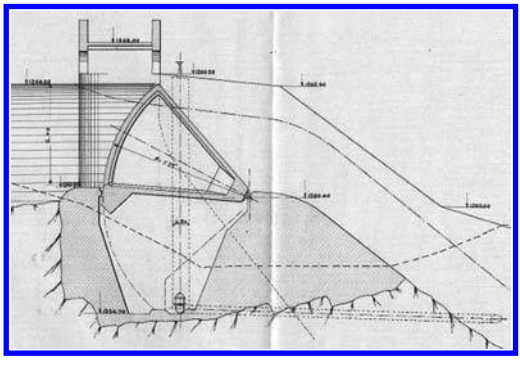

Figura 2. Camporredondo. Sección de la compuerta primitiva del aliviadero de la margen derecha.

Figura 3. Camporredondo. Nueva toma intermedia.

aguas abajo del de Compuerto, con una capacidad mayor.

3.4 *Aliviaderos*

Se ha adjudicado en 2009 la sustitución de las dos compuertas de aliviadero, colocando unas nuevas de 12,00 × 4,80 m^2 en el aliviadero de la margen derecha, y otra de 14,00 × 4,40 m^2 en el aliviadero de la margen izquierda, las dos con alojamiento para ataguías y las ataguías correspondientes. Las dimensiones de las compuertas existentes son idénticas para la de la margen derecha, y de 14,00 × 4,00 m^2 las de la margen izquierda.

4 CERVERA

4.1 *Tomas intermedias y desagües de fondo*

En 1988 se terminó la sustitución de las tomas intermedias, aprovechando el blindaje aguas arriba de la válvula de seguridad.

En la toma intermedia superior se colocó una válvula de seguridad de sección circular de 800 mm de diámetro, y otra de regulación Howell-Bunger también de 800 mm de diámetro.

En la toma intermedia inferior se colocaron dos válvulas de compuerta de 800 mm de diámetro.

En el desagüe de fondo se colocaron nuevos blindajes y dos válvulas Bureau de 1.000 × 1.200 mm^2.

Todos estos trabajos se realizaron vaciando el embalse.

5 BARRIOS DE LUNA

5.1 *Tomas intermedias*

En 1989 se terminó la sustitución de las válvulas de regulación de las dos tomas intermedias. Las anteriores eran Howell-Bunger, de 1.400 mm de diámetro. Las nuevas también son Howell-Bunger, de 1.200 mm de diámetro.

Al haberse cambiado solamente las válvulas de regulación, la obra se realizó sin alterar el régimen del embalse.

6 CUERDA DEL POZO

6.1 *Tomas intermedias y desagües de fondo. Ataguías en compuertas del aliviadero*

En 1995 se terminó la sustitución de las tomas intermedias y los desagües de fondo, y se han colocado alojamientos para ataguías las compuertas del aliviadero.

Las dos tomas intermedias tenían una válvula de seguridad de mariposa de 1.500 mm de diámetro, y un sistema de válvulas de compuertas diseñado con la idea de soltar una gama de caudales parciales con compuertas totalmente abiertas.

Se colocaron blindajes nuevos, ataguías y rejillas. Como válvulas de seguridad se colocaron válvulas de mariposa de 1.500 mm de diámetro, y como válvulas de regulación compuertas Taintor de 900 × 900 mm^2. Fue necesario construir un nuevo cuenco amortiguador y la caseta, en donde se alojan todas las válvulas, previa demolición de todo lo existente.

Los desagües de fondo eran cuatro, agrupados dos a dos en dos edificios, con válvulas de compuerta de seguridad de 1.500 mm de diámetro y un sistema de conductos y válvulas de compuerta de diámetros inferiores para conseguir soltar caudales parciales a sección llena. También tenía ataguías en el paramento de aguas arriba, de 2.500 mm de diámetro.

Una de los grupos de desagüe fue anulado. En el otro se colocaron blindajes nuevos, con escudo. En cada conducto se instalaron dos válvulas Bureau de 700 × 1.000 mm^2.

En los tres vanos del aliviadero, con compuertas Taintor de 12,00 × 6,77 m^2, se instalaron alojamientos para unas ataguías. Se construyeron las ataguías para uno de los vanos, y un sistema de transporte y colocación desde la zona en la que están acopiadas.

7 SANTA TERESA

7.1 *Desagües de fondo*

En 1995 se ha terminado la colocación de nuevas válvulas en los desagües de fondo, mediante escudo.

La presa tiene cuatro conductos de desagües de fondo, agrupados en dos cámaras.

En cada conducto se han colocado dos compuertas Bureau de 1.000 × 1.500 mm^2. También se ha colocado nuevo blindaje en todos los conductos entre los paramentos de aguas arriba y aguas abajo.

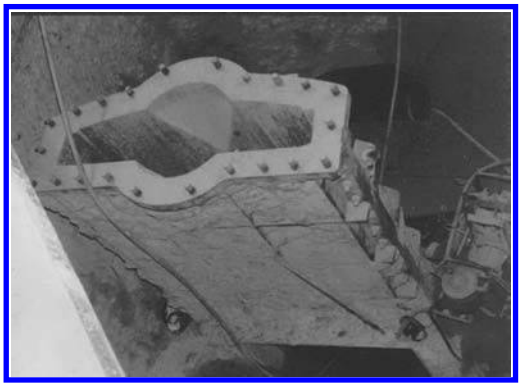

Figura 4. Santa Teresa. Carcasa de una compuerta antigua.

8 ARLANZÓN

8.1 *Tomas intermedias y desagües de fondo*

En 1996 se terminó la sustitución de todos los elementos de las tomas intermedias y los desagües de fondo, coincidiendo con una reparación general de la presa, que afectó al aliviadero, a los paramentos de aguas arriba y aguas abajo, y que incluyó la perforación de una galería perimetral. Todos estos trabajos e hicieron con el embalse vacío, y fue posible por la puesta en carga en esas fechas de la presa de Úzquiza, con un volumen de embalse mucho mayor, situado inmediatamente aguas abajo de la presa de Arlanzón.

La presa tenía tres tomas intermedias, idénticas entre sí, cada una con una válvula de seguridad de 800 mm de diámetro, y una válvula de regulación Larner-Johnson también de 800 mm de diámetro. La superior se anuló, en la segunda se colocaron rejillas, la anterior no tenía, el conducto de 800 mm de diámetro que atravesaba el cuerpo de presa se conservó, tratándolo contra la corrosión, y aguas abajo, inmediata al paramento de la presa se colocó una válvula de seguridad tipo Bureau de 600×800 mm^2 alojada en una caseta, continuando con una conducción de 72 m de longitud que termina en otra caseta en donde se alojan otra válvula de seguridad idéntica a la anterior, y una válvula de regulación Howell-Bunger de 800 mm de diámetro.

En los desagües de fondo se colocaron rejillas, no las tenía anteriormente, y se colocaron todos los blindajes entre los dos paramentos, con su abocinamiento aguas arriba y transición con trampolín aguas abajo. También se colocaron cuatro válvulas Bureau, dos en cada conducto, de 800×1.200 mm^2 de sección. Los anteriores tenían cuatro válvulas Bureau de 1.000×1.500 mm^2 de sección.

Figura 5. Arlanzón. Válvula de seguridad en toma intermedia.

Figura 6. Arlanzón. Válvula Howell-Bunger.

9 SAN JOSÉ

9.1 *Aliviadero*

La presa de San José tenía 8 compuertas Wagon de $15{,}00 \times 5{,}00$ m^2, más otra con el umbral más bajo de $6{,}00 \times 6{,}00$ m^2.

En 1996se construyeron dos juegos de ataguías para las compuertas grandes y uno para la pequeña. Se prolongó el puente grúa para poder cargar las ataguías en la zona habilitada al efecto, y se sustituyeron todas las compuertas por otras de las mismas características y dimensiones.

10 AGUILAR DE CAMPOO

10.1 *Válvulas de regulación de las tomas intermedias*

En 2001 se terminó la sustitución de las dos válvulas de regulación de las tomas intermedias, que eran tipo Larner-Johnson de 1.200 mm de diámetro, colocando válvulas Howell Bunger de 1.200 mm de

Figura 7. Aguilar de campoo. Válvulas Larner-Johnson sustituidas.

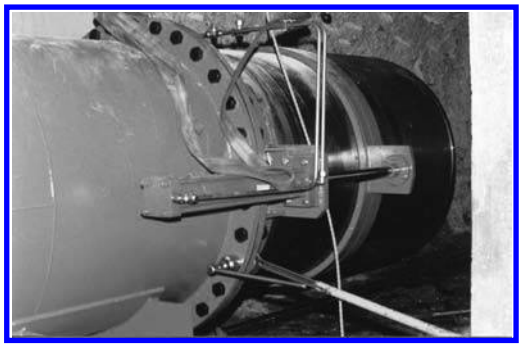

Figura 8. Aguilar de campoo. Nueva válvula Howell-Bunger.

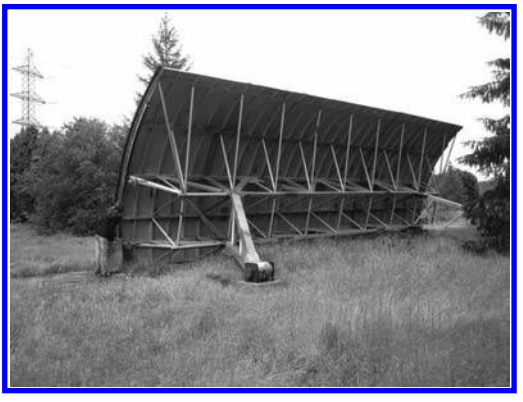

Figura 9. Aguilar. Compuerta de aliviadero sustituida.

Figura 10. Aguilar. Nueva compuerta de aliviadero.

diámetro, con concentrador fijo, y 29 m de blindaje hasta el paramento de aguas abajo.

10.2 *Aliviadero*

En 2004 se terminó la sustitución de las tres compuertas de aliviadero, colocando unas nuevas de $14{,}00 \times 5{,}00$ m^2. También se colocaron alojamientos para ataguiar las tres compuertas, y se suministró un juego de ataguías.

10.3 *Nueva toma intermedia*

Aunque no se trata de la reparación de una toma existente, se cita la construcción de una nueva toma intermedia en la presa de Aguilar.

La toma actual está muy baja. En esa zona la turbidez del agua es alta. Se trata de construir una nueva toma que permitirá tomar agua a dos alturas diferentes.

Se realizarán dos perforaciones en el bloque 25 de la presa a la cota 17 y 28 (la toma actual esta en la cota 5), la primera con escudo, en las que se colocarán sendos conductos que se unirán en la salida al paramento. A partir de ese punto se colocará una conducción enterrada que termina en una caseta en donde se dispondrá una válvula de mariposa y una Howell-Bunger que verterá al colchón del aliviadero. Las dos válvulas son de 1.000 mm de diámetro.

La obra se ha iniciado recientemente.

11 REQUEJADA

11.1 *Válvulas de regulación de las tomas intermedias*

La presa tenía dos tomas intermedias. Cada una constaba de dos conductos paralelos de 800 mm de diámetro. En cada conducto había una válvula de compuerta de seguridad, de 800 mm de diámetro, y en uno de ellos una válvula de regulación Larner-Johnson de 800 mm de diámetro. Es decir, que en un conducto solamente tenía una válvula, pero tenía sitio para montar otra.

Figura 11. Requejada. Válvula Larner-Johnson sustituida.

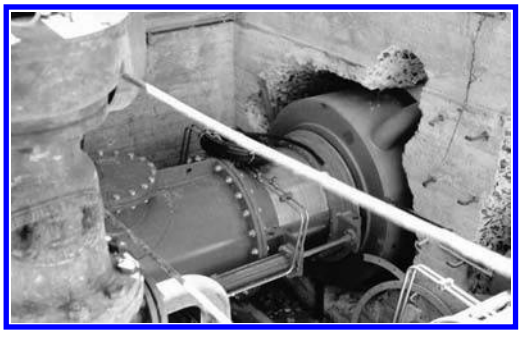

Figura 12. Requejada. Nueva válvula Howell-Bunger.

En 2001, conservado las válvulas de seguridad se terminó la colocación de dos válvulas de regulación Howell Bunger de 800 mm de diámetro en cada toma intermedia, dos de ellas sustituyendo a las Larner-Jonson anteriores, todas con concentrador fijo.

12 VILLAMECA

12.1 *Tomas intermedias, desagües de fondo y compuertas de aliviadero*

En 2001 se han sustituido las compuertas del aliviadero, colocando dos nuevas compuertas

Figura 13. Villameca. Descarga de las compuertas de aliviadero.

Figura 14. Compuerta de aliviadero.

Taintor de 12,00 × 2,10 m^2 en sustitución de las anteriores. Las dimensiones de las que había anteriormente eran de 12,00 × 2,00 m^2.

En las tomas intermedias, que son dos, cada una con un conducto, en cada una se ha colocado una válvula de seguridad tipo Bureau de 600 × 800 mm^2, y una válvula de regulación tipo Howell-Bunger de 800 mm de diámetro con concentrador fijo. Las anteriores eran Larner-Johnson también de 800 mm de diámetro. Esta sustitución se hizo bajando el nivel del embalse.

13 VILLAGONZALO

13.1 *Aliviadero*

Terminada en 2007, la obra ha consistido en la sustitución de las ocho compuertas Taintor existentes por otras nuevas de 15,00 × 15,35 m^2, la rehabilitación del puente grúa y la reparación de los alojamientos de las ataguías. Las compuertas anteriores eran de las mismas dimensiones.

Figura 15. Villagonzalo. Nuevas compuertas de aliviadero.

Figura 16. Águeda. Ataguía colgada del pórtico.

Figura 17. Colocación de una compuerta del aliviadero.

14 ÁGUEDA

14.1 *Aliviadero y desagües de fondo*

Se ha terminado recientemente la sustitución de las compuertas del aliviadero, colocando tres nuevas compuertas Taintor de 15,00 × 4,75 m^2, ataguías y mecanismo para accionamiento de las ataguías. La dimensiones de las que existían anteriormente eran idénticas.

También se ha colocado cuatro compuertas Bureau de 800 × 1.200 mm^2 en los desagües de fondo, y los blindajes completos de los mismos, inclusive los abocinamientos aguas arriba, con la presa en carga, habiéndose utilizado para ello escudos. Las compuertas anteriores eran Bureau de 1.000 × 1.500 mm^2.

15 COMPUERTO

15.1 *Aliviadero, tomas intermedias y desagües de fondo*

Las obras se han adjudicado en 2010.

En el aliviadero se van a colocar dos nuevas compuertas Taintor de 11,50 × 4,00 m^2, con las ataguías correspondientes. Las dimensiones de las compuertas actuales son idénticas.

En las tomas intermedias, que tienen dos conductos, se van a colocar en cada uno dos válvulas de seguridad Bureau de 800 × 1.000 mm^2, y una válvula de regulación Howell-Bunger de 800 mm de diámetro, y los blindajes entre los dos paramentos. En la actualidad hay cuatro compuertas Bureau de 800 × 1.000 mm^2.

En los desagües de fondo, que también tienen dos conductos, se colocarán en cada uno dos válvulas Bureau de 1.000 × 1.500 mm^2, y los blindajes entre los dos paramentos. En la actualidad hay cuatro compuertas Bureau de 1.000 × 1.500 mm^2.

16 LINARES DEL ARROYO

16.1 *Aliviadero, tomas intermedias y desagües de fondo*

Las obras se han adjudicado en 2010.

En el aliviadero se van a colocar dos nuevas compuertas Taintor de 15,00 × 4,50 m^2, con las ataguías correspondientes. Las dimensiones de las existentes son idénticas.

En las tomas intermedias, que va a conservar el conducto existente, colocando en el mismo una válvula de seguridad Bureau de 600 × 800 mm^2, y una válvula de regulación Howell-Bunger de 700 mm de diámetro, con concentrador fijo. Actualmente tiene una válvula de compuerta circular de 800 mm de diámetro aguas arriba, y Larner-Johnson también de 800 mm de diámetro aguas abajo.

En los desagües de fondo, que también tienen dos conductos, se colocarán en cada uno dos válvulas Bureau de 1.000 × 1.600 mm^2. Las dimensiones de las existentes son idénticas.

Las obras se realizarán con escudo.

17 COSTE DE LAS OBRAS

En la tabla que figura a continuación se ha reflejado las características más significativas de las presas, y el coste de las obras realizadas. Como se ha dicho anteriormente el coste está en euros de 2010, actualizando el coste primitivo aplicando los Índices al Precio del Consumo del Instituto Nacional de Estadística.

Tabla 1. Características de las presas y coste de las obras (Millones de euros con dos decimales).

Embalse	Año	Volumen de embalse hm^3	Altura sobre cimientos m	Longitud de coronación m	Aliviadero		Tomas intermedias		Desagües de fondo	
					Año	M€	Año	M€	Año	M€
Camporredondo	1930	70	75	160	1974	0,36	2001	1,61	1956	0,13
					Adj.	2,82			2001	3,22
Cervera	1923	10	36	130			1988	0,28	1988	0,43
Barrios de Luna	1956	308	90	220			1989	0,31		
Cuerda del Pozo	1941	249	40	425			1995	0,98	1995	1,12
Santa Teresa	1960	496	58	527					1995	3,26
Arlanzón	1933	22	47	267			1996	0,55	1996	0,49
San José	1941	6	9	156	1997	5,80				
Aguilar de Campoo	1964	247	48	500	2004	2,10	2001	0,66		
							Ejec.	2,35		
Requejada	1942	65	57	200			2001	0,88		
Villameca	1947	20	39	173	2001	1,22	2001	0,62		
Villagonzalo	1964	6	10	146	2007	7,00				
Águeda	1931	22	38	196	2010	2,58			2010	2,39
Compuerto	1961	95	78	273	Adj.	1,11	Adj.	1,39	Adj.	1,45
Linares del Arroyo	1951	58	36	112	Adj.	1,23	Adj.	0,42	Adj.	0,94
				Suma		24,22		10,05		13,43
									Total	47,70

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Rehabilitación de los órganos de cierre en el desagüe de fondo de la presa de Mansilla y su problemática

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C.H. Ebro

Julio Terrazas Llorente
Einar

Jorge García Orna
Inhisa

RESUMEN: El artículo describe las actuaciones previstas para la sustitución de los elementos de cierre en el desagüe de fondo de la presa de Mansilla. La particularidad del desagüe de fondo de la presa de Mansilla estriba en el aprovechamiento del túnel de desvío del río en la fase de construcción de la presa como desagüe de fondo. Existe por este motivo una gran distancia entre la embocadura y los elementos hidromecánicos situados en la cámara de válvulas añadido a las dificultades de obturación por medio de escudos en embocaduras irregulares y asimétricas en su eje longitudinal y transversal.

ABSTRACT: This article describes the predicted actions in order to repare the sluice gates of the outlet of Mansilla dam. The particularity of the outlet of Mansilla dam is the use of the diversion tunnel used during the construction of the dam as the outlet of the dam. That´s why there is a great distance between the entrance of the tunnel and the gates situated in the gates chamber and also it is very difficult to close this irregular in both longitudinal and cross axis entrance in order to work inside.

1 INTRODUCCIÓN

1.1 *Antecedentes*

La presa de Mansilla está situada en el término municipal de Mansilla de la Sierra en la provincia de La Rioja, sobre el río Najerilla, afluente del río Ebro por su margen derecha.

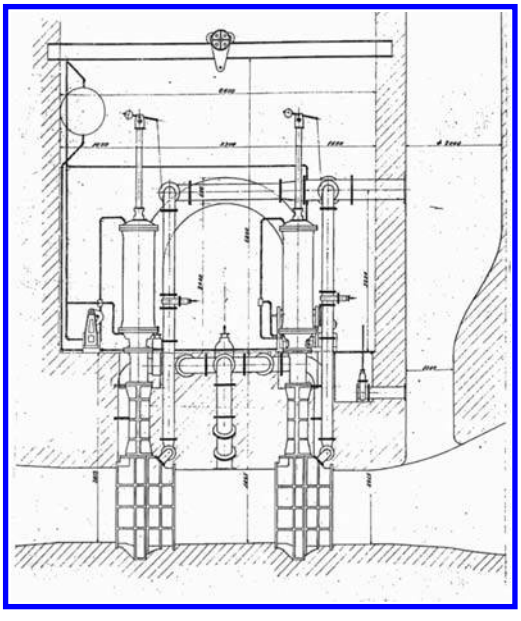
Fue terminado de construir en 1960 y tiene una capacidad de 67,70 Hm^3. con N.M.N. a la cota de 930 m.s.n.m. La presa es de gravedad de hormigón de planta recta con una altura sobre cimientos de 81,80 m.

El desagüe de fondo está formado por dos conductos paralelos situados en el túnel utilizado como desvío del río durante la construcción de la presa. Este túnel discurre bajo el estribo derecho de la presa y el acceso a la cámara de maniobra se realiza a través de una galería a la que se accede desde aguas abajo de la presa por su margen derecha.

Cada conducto dispone de dos compuertas tipo Bureau y la capacidad máxima de desagüe es de 75 m^3/s por conducto. La carga de agua en los cierres es de aproximadamente 70 m.c.a.

Las compuertas bureau fueron construidas por el constructor BOETTICHER Y NAVARRO con unas dimensiones de paso de 1,25 × 1,50 mts. Las compuertas de seguridad y las de regulación son gemelas y se encuentran hormigonadas hasta su cúpula.

Por otro lado el desagüe de fondo no dispone de rejas, ataguias, blindajes o tuberías, lo que dificulta



más si cabe cualquier solución. En un primer momento se desconocía el estado de la embocadura ignorando si estaba aterrada o no.

2 PROBLEMÁTICA DE LA SUSTITUCION DE LOS ORGANOS DE CIERRE DEL DESAGÜE DE FONDO

2.1 *Generalidades*

En la actualidad, las labores para la sustitución de los órganos de cierre se encuentran suspendidas hasta que la altura del agua permita la reanudación de los trabajos subacuaticos.

La necesidad de realizar actuaciones sobre los elementos de cierre viene dado por el desuso que se hace desde hace años de las compuertas de seguridad del desagüe de fondo y por la existencia de fugas del orden de 2 m^3/s tanto en estas compuertas como en las de regulación. En su momento se estudiaron dos soluciones distintas, una que abarcaría la rehabilitación total de todas las compuertas mediante su reparación, por lo que se hace necesario colocar escudos aguas arriba de las compuertas de seguridad con el fin de poder acceder en seco para proceder a su inspección, desmontaje y reparación.

Otra solución y solo en caso de imposibilidad de colocación de los escudos, seria, aprovechar las compuertas de aguas arriba como ataguias las cuales una vez selladas dejarían el conducto en seco pasando las primitivas compuertas de regulación a ser las de seguridad, construyendo unas nuevas compuertas de regulación aguas abajo de estas.

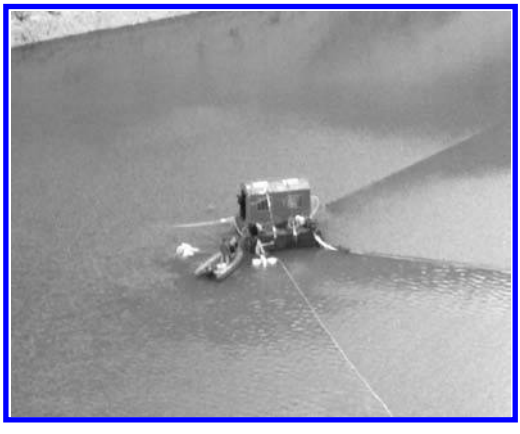
2.2 *Solución adoptada*

Se opta por la primera solución de mantener el esquema inicial de compuertas y confiando en vencer las incertidumbres que representaba la colocación de unos escudos en una galeria de 100 metros de longitud, en estado desconocido y con fugas existentes en las compuertas que era necesario tapar para poder acceder en condiciones de seguridad con medios subacuaticos.

Las actuaciones seguidas son las siguientes:

Por un lado se ha procedido localizar la posible embocadura del túnel de desvío mediante la movilización de buzos y pontonas, siendo esta balizada para las posteriores actuaciones.

Para garantizar la seguridad de operación de los buzos, se perfora verticalmente la presa desde un estribo en el cuerpo de presa con el fin de localizar



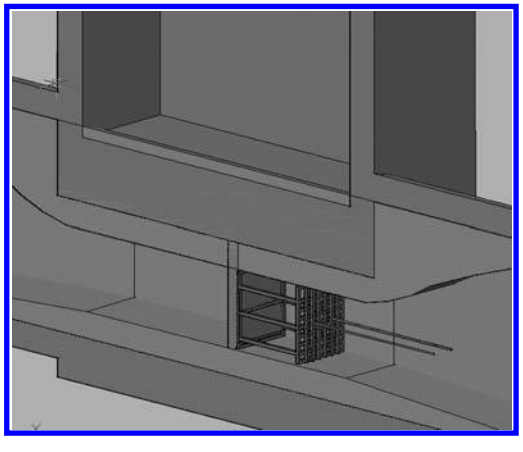
un punto aguas arriba de las compuertas de seguridad y poder introducir material de sellado con la intención de reducir las fugas en lo posible y no provocar la posible absorción de los buzos cuando se encuentren en la zona de las embocaduras próxima a las compuertas. Parcialmente se reducen las fugas y se complementa con el sellado de las compuertas de regulación.

Una vez selladas las fugas, en una primera inspección se confirma que el túnel de acceso se encuentra aterrado y solo en su tramo final cerca de la embocadura donde se convierte en dos conductos rectangulares es donde este aterramiento disminuye.

Esta situación crea la obligación de diseñar y calcular unos escudos que dentro del agua se encuentren equilibrados y pudieran ser manipulados con facilidad. A continuación se realiza una inspección en profundidad de la zona de cierre de los escudos para lo cual se construyen unos gálibos a fin de comprobar las dimensiones y geometría definitiva.

Para que estos escudos pudieran trabajar correctamente se diseñan unas ménsulas de apoyo que son las encargadas de transmitir las cargas del escudo al hormigón de los conductos, ancladas estas por medio de anclajes de alta resistencia.

Antes de la instalación de las ménsulas se procede a la comprobación mediante otro galibo de la situación exacta para la colocación del escudo y la comprobación final de las dimensiones del mismo.



Debido al rápido aumento del nivel de embalse durante el último tramo del año que impidió culminar los trabajos de colocación de los escudos, en especial en lo tocante al número de anclajes necesarios para soportar la carga de agua, y debido también a las bajas temperaturas que hacen imposible el trabajo a gran profundidad, se paralizan los trabajos hasta que el agua permita la colocación del resto de anclajes, presumiblemente en septiembre de 2010.

3 CONCLUSIONES

Como conclusión cabe destacar que dadas las dificultades técnicas encontradas, las variables a contemplar y las condiciones meteorológicas adversas encontradas durante las primeras fases de estos trabajos, no ha sido posible hasta la fecha la colocación de los escudos. Una vez estos colocados se procederá a la reparación de las compuertas del desagüe de fondo de la presa de Mansilla.

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Estudio de protecciones frente al sobrevertido de presas de materiales sueltos mediante repié de escollera

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RESUMEN: El contenido del artículo se enmarca dentro de la línea de investigación iniciada hace años por la unidad docente de presas de la ETSI de Caminos, Canales y Puertos de la Universidad Politécnica de Madrid (en adelante UPM), referente al estudio del vertido por coronación (también denominado sobrevertido u *overtopping*, en inglés) en presas de escollera. Este tema está adquiriendo importancia a nivel mundial y prueba de ello es que importantes organismos a nivel mundial están investigando y redactando manuales de protecciones de presas frente al sobrevertido. El objetivo principal de la investigación es estudiar el efecto de las protecciones de escollera sobre la seguridad de la presa en situación de sobrevertido, analizando la influencia de su geometría y tipo de material utilizado, con el fin de definir criterios de dimensionamiento de este tipo de protecciones que eviten la rotura de la presa por deslizamiento en masa. El trabajo objeto del artículo se basa en una serie de ensayos en modelo físico realizados en el Laboratorio de Hidráulica de la ETSI de Caminos, Canales y Puertos de la UPM con motivo del trabajo de investigación tutelado llevado a cabo en 2008 dentro del Departamento de Ingeniería Civil: Hidráulica y Energética.

ABSTRACT: The content of the article is part of the research begun years ago by the Dam's teaching unit of the ETSI de Caminos, Canales y Puertos in the Polytechnic University of Madrid (UPM), concerning the overtopping study in rockfill dams. This issue is becoming important worldwide and national dam safety agencies are investigating about protections against overtopping in dams, especially in embankment and rockfill dams, which are more vulnerable to this phenomenon. The main objective of this research is to study the effect of the rockfill protections on dam safety in overtopping scenarios. To do this, it has to be analyzed the influence of the geometry and type of material. In future investigations, criteria for the protection design to prevent mass slide failure will be defined. The work covered by the article is based on a campaign of physical model tests conducted at the Laboratory of Hydraulics of the ETSI de Caminos, Canales y Puertos of the UPM on the occasion of a pre doctoral research work conducted in 2008 within the Department of Civil Engineering: Hydraulics and Energy.

1 INTRODUCCIÓN

El trabajo descrito se basa en una serie de ensayos en modelo físico. Estos ensayos no están planteados como modelos a escala de un caso real sino que son en sí mismos el prototipo de estudio. Por lo tanto, el análisis dimensional y los denominados como efectos de escala, cuyo análisis corresponde a otro trabajo de investigación que se encuentra en ejecución, no son objeto de estudio. No obstante, las características geométricas y materiales utilizados en los ensayos están dentro de los rangos habitualmente manejados en la construcción de presas.

Como ya se ha adelantado, la protección a estudiar consiste en un repié de escollera, formado por una berma aguas abajo del espaldón de la presa principal de la misma anchura que la coronación de la presa (20 cm), rematada mediante un talud aguas abajo. En la campaña únicamente se han variado dos parámetros de la geometría de la protección: la altura de la berma (H_b) con relaciones H_b/H (siendo H la altura de la presa principal) de 0.6 y 0.4 y el talud de aguas abajo (N_b) con valores 1.5; 2.2 y 3.0. El material, también granular, es de 4.5 cm de tamaño medio.

Adicionalmente, se analiza el comportamiento de la presa sin protección alguna para comparar los daños de la presa protegida y sin proteger. Este caso, unido a los seis anteriores, hace que el número total de ensayos realizados ascienda a siete.

2 DESCRIPCIÓN DE LA CAMPAÑA DE ENSAYOS EXPERIMENTALES

2.1 *Instalación de ensayo*

El recinto de ensayo se encuentra ubicado en el laboratorio de Hidráulica de la ETSI de Caminos, Canales y Puertos de la Universidad Politécnica de Madrid y tiene unas dimensiones de 2.4 m de anchura, 1.4 m de altura y 13.7 m de longitud. Se trata de una obra de fábrica con solera horizontal, dotada de una ventana de inspección lateral de cristal en su cajero izquierdo, con dos lunas de 2.30×1.08 m^2, cada una de ellas.

2.2 *Descripción de los modelos físicos*

La sección de la presa y sus protecciones se mantiene constante a lo ancho del canal de ensayo. Por tanto, el estudio se limita a casos bidimensionales.

La dimensión relativa de la anchura del canal (W) respecto de la altura de la presa es el factor crítico a la hora de evitar el efecto pared. En la campaña, este factor se mantiene constante W/H = 2.4. Dado que el comportamiento del modelo queda distorsionado en las proximidades de las paredes del recinto se ha forzado que la rotura se produzca alejada de dichas paredes. Esto se ha conseguido colocando tela metálica junto a ellas para dificultar el movimiento de las piedras en esa zona.

La geometría de la presa que es objeto de protección en el ensayo se mantiene invariable. Se trata de un cuerpo de presa de taludes M = 1.5 y N = 1.5 aguas arriba y aguas abajo, respectivamente, con una anchura de coronación C de 0.20 m y una altura H de 1 m. Se han elegido taludes del mismo orden de magnitud que los que se usan habitualmente en las presas de escollera.

El elemento impermeabilizante, también invariable en esta campaña, simula una pantalla externa y está formado por una lámina de PEAD de 1.5 mm montada sobre un marco de aluminio que descansa sobre el talud de aguas arriba y sellado en el contacto con cajeros y solera mediante una masilla elastomérica.

El caso de la presa sin protección ($H_b/H = 0$) se ha ensayado como caso de referencia para poder analizar y comparar los daños con respecto a la presa protegida.

La geometría de la protección se ha caracterizado mediante dos parámetros cuya influencia en el fenómeno es mayor, esto es, el talud N_b y la altura de la protección H_b (Fig. 1). Para facilitar el tratamiento de los datos obtenidos se ha optado por trabajar con la variable adimensional $H_b^* = H_b/H$. El ancho de la berma se ha mantenido constante, de 20 cm.

El rango de variación de la altura de la protección viene delimitado fundamentalmente por dos factores. El primero es el tamaño medio del material de protección, que la limita inferiormente con el objetivo de asegurar un funcionamiento asimilable a un continuo. La relación entre el tamaño de la partícula y las dimensiones del conjunto debe estar limitada para que los efectos de imbricación y colocación relativa entre las partículas no influyan en exceso. *De Cea y Olalla* (1991) aconsejan que esta relación sea como mínimo 6. En los ensayos se ha limitado la relación entre el tamaño de partícula y la altura de protección (d_{50b}/H_b) a 8.

El segundo factor que se ha tenido en cuenta a la hora de plantear la altura máxima de la protección es la zona de salida prevista para el caudal de filtración del espaldón. Tal y como señala Toledo en el cálculo de la ley de intercambio de caudal, con los taludes habitualmente utilizados en presas de escollera (rango entre 1.5 y 2.0) dicha zona de salida se limita a aproximadamente el 60% de la

Figura 1. Vista lateral de la protección colocada antes de iniciar el ensayo.

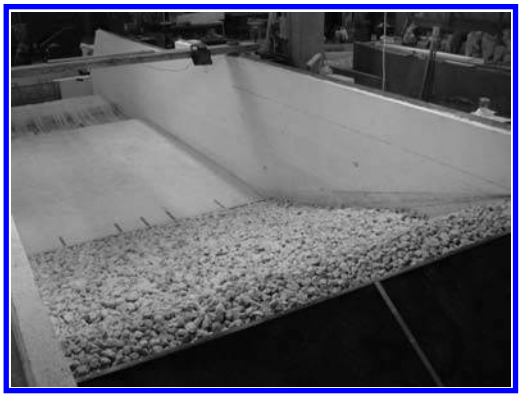

Figura 2. Vista desde aguas arriba de la disposición del modelo con la impermeabilización con lámina de PEAD.

longitud del talud (*Toledo 1997*) y por lo tanto de la altura de presa. Por ello se adopta como límite superior $H_b^* = 0.6$. Además, la zona con presiones relativas más altas está por debajo de esa altura, con lo que la estabilidad frente al deslizamiento queda garantizada por encima de ella.

Los taludes del repié (N_b) utilizados en el ensayo son 1.5; 2.2 y 3.0. El objeto es analizar un rango de taludes amplio para poder extraer conclusiones en función del mecanismo de rotura que predomine, bien el deslizamiento en masa, mayor en taludes bajos, o bien la erosión por arrastre de las partículas, predominante cuando la estabilidad en masa está garantizada.

En resumen, los casos a ensayar desde el punto de vista de la geometría son los siguientes:

- Caso $H_b^* = 0$. Presa sin protección
- Caso $H_b^* = 0.6$; $N_b = 1.5$
- Caso $H_b^* = 0.6$; $N_b = 2.2$
- Caso $H_b^* = 0.6$; $N_b = 3.0$
- Caso $H_b^* = 0.4$; $N_b = 1.5$
- Caso $H_b^* = 0.4$; $N_b = 2.2$
- Caso $H_b^* = 0.4$; $N_b = 3.0$

El caudal máximo disponible en la instalación está limitado a unos 95 l/s con la disposición actual del sistema de alimentación del recinto de ensayo. La serie de caudales a ensayar se organiza en 6 escalones como máximo, parando el ensayo una vez que se produce la rotura total de la protección. El caudal se incrementa bruscamente en cada escalón, manteniéndolo constante durante un mínimo de 20 minutos. El criterio para iniciar el cambio de escalón es que no se produzca movimiento de piedras en 10 minutos consecutivos. Los caudales de los escalones se aumentan de forma diferente en función de cada caso, tratando de mantener como incremento medio los 15 l/s.

2.3 *Datos obtenidos*

Las lecturas se realizan inmediatamente antes del cambio de escalón de caudal.

2.3.1 *Caudal*

El caudal de alimentación es un parámetro fundamental, por ello se ha medido por 3 procedimientos distintos.

1. Mediante caudalímetro de ultrasonidos ubicado en la tubería de alimentación.
2. Mediante un dispositivo tipo Venturi instalado en dicha tubería.
3. Mediante una sonda de nivel aguas arriba del vertedero situado a la salida del canal de ensayo.

2.3.2 *Calados en el canal*

Se han medido los niveles aguas arriba de la presa y en el extremo de aguas abajo del canal, antes de la entrada del agua al desarenador. Para ello se utilizan dos sondas de nivel situadas en estos emplazamientos.

2.3.3 *Presiones en la base*

Se han realizado lecturas de los piezómetros situados a lo largo de la base ocupada por el material de presa y protección. Se han tomado las medidas de la línea más cercana del eje longitudinal del canal.

2.3.4 *Línea de saturación*

Se han tomado medidas de la altura de la línea de saturación en el muro izquierdo del recinto, haciendo uso de la ventana de cristal situada en el cajero izquierdo. Las medidas se han realizado directamente mediante una escala milimétrica. Estas medidas se han utilizado como comprobación tras la calibración del modelo numérico de filtración. Se han tomado medidas en los siguientes puntos.

- En el remanso aguas arriba de la coronación de la presa, en la vertical del extremo de aguas arriba de la coronación.
- En los puntos que definen los extremos de la berma de la protección.
- En el punto donde la línea llega a la superficie libre.

2.3.5 *Altura de los daños producidos*

Una vez producidos los daños sobre la protección se ha medido la altura, medida desde el fondo del canal, hasta donde ha progresado la erosión o deslizamiento sobre el espaldón.

La rotura se considera en el momento en que la brecha o deslizamiento llega a la coronación de la presa (rotura de la presa). Se considera rotura de la protección cuando los daños alcanzan la berma

Figura 3. Ensayo de protección con talud 3 (Nb = 3) y altura del 40% de la altura de la presa (Hb* = 0.4). Nivel de daño 1.

de la protección o bien cuando el material de la presa comienza a verse afectado.

3 RESULTADOS

Los caudales unitarios que han sido soportados por las protecciones son los siguientes, expresados adimensionalmente con respecto al caudal que rompe completamente la presa sin protección, son los siguientes.

Figura 4. Ensayo de protección con talud 2.2 ($N_b = 2.2$) y altura del 40% de la altura de la presa ($H_b^* = 0.4$). Nivel de daño 3.

Para un nivel de daño 1, definido como el estado en que se ha producido una recolocación de las piedras superficiales con algún movimiento de piedras singulares, sin variación de la geometría apreciable:

Tabla 1. Caudales unitarios adimensionales que soporta la protección con nivel de daño 1, en función del talud (N_b) y de la altura adimensional (H_b^*/H) de la protección.

	$N_b = 1.5$	$N_b = 2.2$	$N_b = 3.0$
$H_b^*/H = 0.4$	0.35	0.56	1.10
$H_b^*/H = 0.6$	0.67	0.61	1.10

El caudal unitario máximo de alimentación es 1.1. La aparición de este resultado indica que no se pudo llegar al caudal límite soportado por la protección, al quedar limitados por el caudal de alimentación del laboratorio.

Para un nivel de daño 3, definido como el estado en que se ha producido la rotura de la protección o bien cuando se empieza a ver afectado el material del espaldón de la presa a proteger:

Tabla 2. Caudales unitarios adimensionales que soporta la protección con nivel de daño 3, en función del talud (N_b) y de la altura adimensional (H_b^*/H) de la protección.

	$N_b = 1.5$	$N_b = 2.2$	$N_b = 3.0$
$H_b^*/H = 0.4$	0.67	0.92	1.10
$H_b^*/H = 0.6$	0.88	1.09	1.10

4 CONCLUSIONES

Las conclusiones se pueden resumir en los siguientes epígrafes:

1. Las protecciones ensayadas permiten mejorar sensiblemente la estabilidad frente al deslizamiento en masa provocado por el sobrevertido en presas de escollera. Las condiciones para este resultado han sido:
   a. La granulometría de las protecciones garantizaba la estabilidad interna del material.
   b. El tamaño del material seleccionado para la protección evitaba el arrastre para los caudales unitarios disponibles en la alimentación del canal.
   c. La permeabilidad (con ley de resistencia lineal) del material de protección era 1.7 veces mayor que la de la presa.
   d. El ángulo de rozamiento del material de presa es de 40º mientras que el de la protección se acerca a los 43º.
2. Sólo se admitió el deslizamiento dentro del material de protección por lo que el ángulo de rozamiento movilizado ha sido siempre el de ésta última. El ensayo es representativo por tanto siempre que el ángulo de rozamiento de la presa sea tal que no lleguen a formarse círculos de rotura en su interior.
3. Los volúmenes de material de protección por metro lineal necesarios para conseguir un nivel de daños moderado y resistir caudales que romperían la presa en caso de "no protección" están en el rango del 10 al 15% de los volúmenes de la presa, también por metro lineal. Con valores entre el 5 y el 10% se llegaba a un nivel de protección menor, resistiendo caudales unitarios de entre el 60% y el 80% del caudal de rotura de la presa. En este análisis, se está suponiendo que la presa y la protección mantienen la misma sección a lo largo de todo el eje de la presa. Esto ofrece unos valores más altos que los reales por lo que, en un caso real, los volúmenes de protección necesarios serán inferiores.
4. El talud $N_b = 1.5$ ha ofrecido resultados muy pobres mientras que el talud $N_b = 3.0$ se ha comportado como un talud estable, incluso en condiciones de saturación completa de la protección.

5. La solución más eficiente de los casos ensayados ha sido: $H_b^* = 0.4$. $N_b = 3.0$. Es la solución que ha tenido menor nivel de daño en comparación con el material que ha sido necesario utilizar para su construcción.

5 INVESTIGACIONES FUTURAS

Las nuevas investigaciones a realizar deberían incluir una mayor cantidad de parámetros a variar y un mayor detalle en los parámetros variados en este trabajo. Lógicamente en muchos casos será necesario apoyarse en modelos numéricos dada la complejidad, el tiempo necesario y el costo de los modelos físicos. Se propone el estudio de:

1. Comportamiento ante la variación del rango de taludes de presa, N. Desde 1.3 a 1.8, cubriendo el rango habitual de la tipología de presa de escollera.
2. Variación del parámetro anchura de la berma de protección, B.
3. Ampliación de los casos de talud de protección Nb, especialmente en el rango entre 3.0 y 2.2.
4. Variación de los parámetros de ángulo de resistencia al corte de la presa y de la protección.
5. Variación del coeficiente de uniformidad de la protección.
6. Variación de la permeabilidad relativa de la protección con respecto a la de la presa.
7. Análisis del efecto de la forma del valle.

Inicialmente se plantea la realización de un estudio paramétrico utilizando modelación matemática. Posteriormente los resultados deberán ser validados mediante modelación física. El objeto final de la investigación es la propuesta de unos criterios concretos de diseño de protecciones de escollera.

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Aliviadero complementario de la presa de Calanda

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ABSTRACT: The outlet capacity of the current Calanda-Dam spillway achieves 1500 m^3/s, which is highly insufficient to comply with dam regulations. In order to solve this problem, two different auxiliary spillways have been projected with the next characteristics 50 meter-diameter and shaft type. This alternative was ruled out, after testing by reduced model, because negative pressures were produced in the upper section of the elbow, which would provoke important structural damages. Different tests were done using reduced as well as mathematical model until a result acceptable was achieved. The final solution consists of two labyrinth spillways with an outlet tunnel, these tunnels allow to drain extreme and designed floods, always flowing as an open channel.

1 INTRODUCCIÓN

La Presa de Calanda, de 1980, se encuentra situada en el río Guadalope, entre las desembocaduras de los ríos Bergantes y Guadalopillo, en el término municipal de Calanda en la provincia de Teruel.

El aliviadero actual de la presa está formado por tres vanos de 10 m de anchura con compuertas Taintor de 5,45 m de altura, cuya capacidad máxima de evacuación es de 1.500 m^3/s.

En 1988 se realizaron dos estudios: "Estudio de seguridad de la Presa de Calanda" y "Estudio de seguridad de avenidas extraordinarias de los embalses del Guadalope". En estos estudios se consideraron series de datos más extensas que las consideradas en proyecto y se dedujo que la presa de Calanda era incapaz de laminar la nueva avenida de 500 años de periodo de retorno.

De acuerdo con el Proyecto de Aliviadero en la Presa de Calanda que se está redactando actualmente, la solución sería construir un nuevo aliviadero capaz de laminar avenidas de 1000 años de periodo de retorno que cumpla con la normativa actual. Según los datos que figuran en dicho proyecto, el caudal total laminado para la Avenida de Proyecto es de 3.450 m^3/s y de 4.680 m^3/s para la Avenida Extrema.

2 DESCRIPCION DE LA OBRA. SOLUCION PROYECTO

La Presa de Calanda es una presa de materiales sueltos con núcleo de arcilla, de 63,50 m de altura sobre cimientos y coronación situada a la cota 443,50.

La cuenca vertiente tiene una superficie de 2.691 km^2 y el embalse tiene una capacidad de 54,31 hm^3 a la cota correspondiente a nivel máximo normal.

La capacidad de desagüe del aliviadero existente en la Presa de Calanda es insuficiente para desaguar las avenidas con elevado periodo de retorno. Los caudales que debe evacuar el aliviadero exigirían una ampliación, pero las características de la presa y de la cerrada obligan a la construcción de un aliviadero complementario.

Si bien en principio los técnicos de la Confederación pensaron en un único aliviadero tipo laberinto, finalmente se decantaron por mantener el aliviadero actual y construir dos aliviaderos complementarios tipo pozo ubicados en el macizo rocoso de la margen derecha.

Los aliviaderos complementarios, tipo pozo, se disponen sobre una plataforma a la cota 430,00. Cada uno de ellos consta de embocadura, codo, zona de aireación, zona de transición y túnel. Los túneles de los dos aliviaderos finalizan en un cuenco de resalto común.

La embocadura de los aliviaderos tiene un perfil tipo Creager calculado para 2,30 m de lámina vertiente correspondiente a la Avenida de Proyecto. El radio del aliviadero es de 25 m, por lo que la longitud desarrollada de cada uno es de 157,08 m. El umbral se instala a la cota 440,00, es decir 5,15 m por encima del umbral del aliviadero actual situado a la cota 434,85.

A partir de la cota 438,70 m se inicia lo que hemos denominado "codo". Este codo es simétrico respecto al eje de simetría de la sección tipo del túnel. Observando la sección por el eje, la definición a partir del final de Creager de la zona de aguas arriba comienza con una recta tangente

a él de 44,92 m a la que sigue una curva de 21, 33 m de longitud y 29,49 m de radio. La zona de aguas abajo queda definida a partir del Creager mediante una recta de 4,69 m seguida de dos curvas, la primera de 15,00 m de radio con una longitud de 23,14 m, y la segunda con un radio de 20,60 m y 10, 98 m de longitud.

La sección de salida del codo queda definida mediante una curva de radio 21,48 m en la solera, otra curva de radio del mismo radio en los hastiales y una recta en la clave como intersección de la sección tórica del codo cortada por una curva en la parte superior de 20,60 m de radio.

La superficie del codo se genera partiendo de la sección por el eje junto con la sección circular en la embocadura, otra sección circular intermedia y la sección de salida. La sección circular intermedia se encuentra situada en la tangencia entre la recta y la curva de la zona de aguas arriba de la sección definida por el eje. El diámetro, 11,74 m, de esta sección queda fijado por su ubicación entre las dos curvas que definen la sección por el eje.

La zona final del codo se define mediante dos curvas con el mismo centro, la parte inferior de radio 29,49 m y la superior de radio 18,75 m. La parte superior se corta por la curva de radio 20,60 m hasta la sección de salida del codo.

Entre el final del aliviadero y el inicio del túnel se encuentran los 40 m de la zona de aireación. En esta zona la sección a la salida del codo del aliviadero se incrementa 0,5 m alrededor completándose por la clave con la sección circular que se ha recortado en la parte final del codo, hasta alcanzar los 117,83 m^2. En la zona de aireación se ubica una tubería de 2,50 m de diámetro que conecta con la atmósfera a través del muro antivórtice situado en el embalse.

Entre la zona de aireación y la sección tipo del túnel se localiza una zona de transición de 10 m de longitud. Desde este punto se inicia el túnel con una sección de 106,66 m^2, una pendiente del 4% y una longitud de 294,29 m. Al tratarse de una pendiente fuerte con circulación del flujo en régimen rápido, el agua se acelera a lo largo del túnel disminuyendo el calado, por lo tanto, no se prevé la entrada en carga a la salida del túnel.

Al final de ambos túneles está proyectada la construcción de un cuenco amortiguador cuya misión es restituir los caudales evacuados al cauce natural disipando la mayor parte de la energía del flujo.

En el modelo físico se ha representado uno de los aliviaderos complementarios con una longitud de 70 m de túnel.

3 ENSAYO DE LA SOLUCIÓN PROYECTO

3.1 *Modelo fisico*

La realización de este estudio se ha efectuado sobre el modelo físico reducido construido para el estudio de la Solución Proyecto con escala geométrica 1/50.

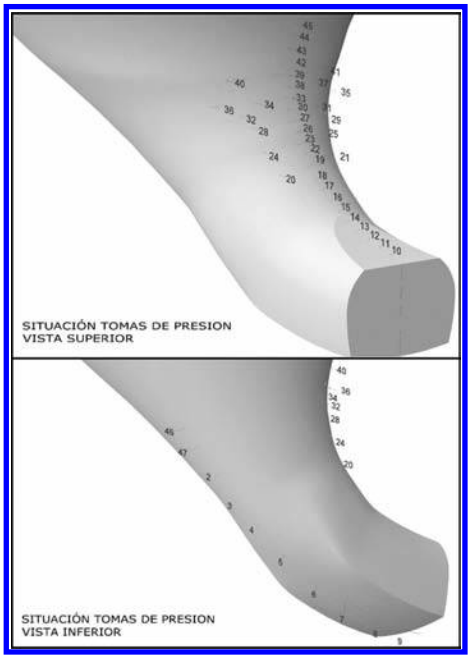

Figura 1. Situación de las tomas de presión.

En este modelo se ha utilizado un limnímetro manual ubicado en la zona del embalse para la obtención de la curva de capacidad y un sistema Scanivalve para la medida de presiones.

El sistema Scanivalve se utiliza para medir las presiones estáticas (en el momento de la medida) en 48 tomas distintas. Para ello dispone de un dispositivo electromecánico que comunica, de manera secuencial, cada toma de presión con un sensor de presión diferencial.

3.2 *Curva de capacidad del aliviadero*

El primero de los trabajos realizados en el modelo consiste en la obtención de la curva de capacidad del aliviadero ensayado. Para ello se han suministrado caudales al modelo que varían entre los 100 y los 2.000 m^3/s.

La curva obtenida experimentalmente proporciona valores de la lámina de agua en el embalse similares a los calculados de forma teórica con la laminación conjunta del aliviadero principal y de los dos aliviaderos de emergencia.

3.3 *Ensayos de la solución proyecto*

Simultáneamente a la obtención de la curva de capacidad se realizaron los ensayos correspondientes a la Solución Proyecto. Durante cada ensayo se ha observado el funcionamiento del aliviadero y

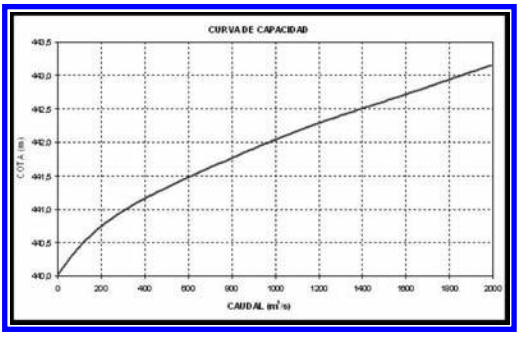

Figura 2. Curva de capacidad de un aliviadero.

registrado las presiones en cada una de las tomas para los diferentes caudales ensayados.

Los ensayos se han realizado para 22 caudales, que fluctúan entre 100 y 2.000 m^3/s con variaciones de 100 m^3/s entre ellos, prestando especial interés en los valores correspondientes a un aliviadero para la Avenida de Proyecto, 1.125 m^3/s y para la Avenida Extrema 1.673 m^3/s.

Tanto en las fotografías tomadas durante los ensayos como en los registros de presión efectuados en los distintos puntos se puede observar un cambio del funcionamiento del aliviadero que tiene lugar entre los 600–900 m^3/s. En los gráficos se aprecia como en muchas de las tomas situadas en la parte superior del aliviadero no se registran presiones hasta superado este caudal, esto es debido a que se encuentran a presión atmosférica (tomas desde la nº 25 a la 45).

3.3.1 *Caudal Q = 300 m^3/s*

Para caudales menores de 300 m^3/s se produce un choque entre las láminas que entran por los dos lados del eje de simetría dando lugar a la formación de un "penacho".

El agua no alcanza la mayoría de las tomas situadas en la parte superior del codo por tanto estas tomas se encuentran a presión atmosférica, no obstante se han registrado algunos valores mínimos negativos de 5 m.c.a. Por el contrario, en general las tomas situadas en la parte inferior se registran presiones positivas alrededor de los 5 m.c.a.

3.3.2 *Caudal Q = 600 m^3/s*

Para un caudal de 600 m^3/s el choque entre láminas es mayor y el "penacho" alcanza la lámina que desciende por la zona opuesta. Esta lámina, que vierte en las proximidades del muro antivórtice, se despega de las paredes del codo donde éste cambia de curvatura.

3.3.3 *Caudal Q = 900 m^3/s*

Con este caudal se empieza a percibir un cambio en el funcionamiento del codo. La lámina vertiente es mayor y se notan perturbaciones a la entrada en las proximidades del muro antivórtice. La influencia de la entrada del flujo en el aliviadero es importante. Se inicia la formación de un remolino en el codo.

En los gráficos de las tomas de presión se observa que en las tomas situadas en la clave de la zona final de codo (nos 10 a 17) existen depresiones. En las tomas de la zona intermedia, entre la nº 18 y

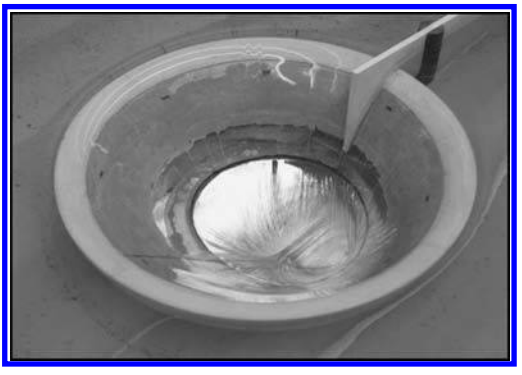

Figura 3. Q = 300 m^3/s. Vista lateral del choque entre láminas.

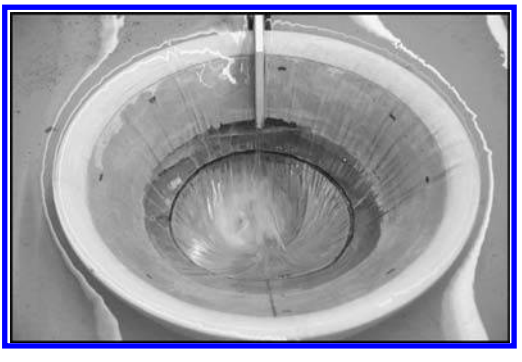

Figura 4. Q = 600 m^3/s. Vista frontal del choque entre láminas.

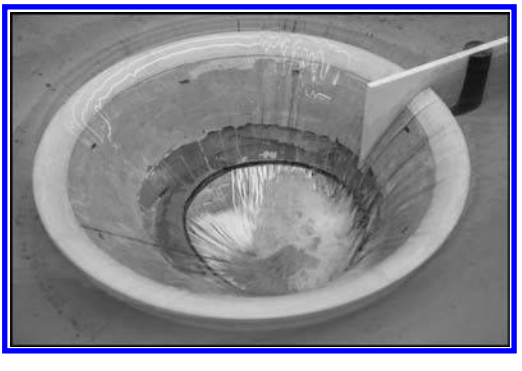

Figura 5. Q = 900 m^3/s. Vista lateral del choque entre láminas.

la 26, las presiones son positivas y entre la toma 27 y la 38, si bien las presiones medias son positivas, se han registrado presiones mínimas con valores negativos. En la zona más alta del codo, entre las tomas 39 y 45, se ha registrado presión atmosférica lo que nos indica que esta zona está aireada. En la zona inferior del codo, entre las tomas 46 y 9 las presiones son mayoritariamente positivas.

3.3.4 *Caudal Q = 1.125 m³/s. Avenida de Proyecto*

El caudal de 1.125 m³/s es el correspondiente a la Avenida de Proyecto para uno de los aliviaderos de emergencia. En las fotografías se aprecia claramente el remolino que se forma en el codo. Este remolino no afecta a los niveles en el embalse, pero si a las presiones registradas. También se puede observar las líneas de corriente al aproximarse al aliviadero, donde se forman pequeñas perturbaciones principalmente en la zona próxima al muro antivórtice.

En los gráficos de las presiones registradas para este caudal se puede distinguir dos comportamientos diferenciados. Por una parte, en las tomas situadas en la parte inferior del codo se registran esencialmente presiones positivas que superan, en sus valores máximos, los 20 m.c.a.

Por el contrario, en las tomas de la parte superior, si bien la presión media de las registradas son todas positivas, en las presiones mínimas se han registrado valores negativos, que pasados a escala, estarían por debajo de los 10 m.c.a., valores que no pueden ser reales al estar por debajo del cero absoluto, pero que indican la presencia de una zona peligrosa.

3.3.5 *Caudal Q = 1.673 m³/s. Avenida extrema*

Durante los ensayos correspondientes al caudal de la Avenida Extrema se ha observado un funcionamiento similar a los caudales anteriores. Se ha podido apreciar la influencia de entrada del flujo en el aliviadero y como los niveles de la lámina de agua a ambos lados del muro antivórtice son distintos.

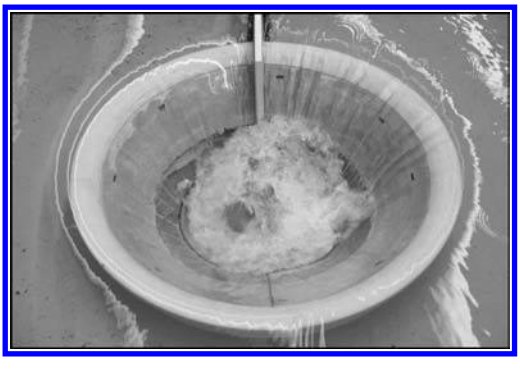

Figura 6. Q = 1.125 m³/s. Vista superior del remolino.

Figura 7. Q = 1.673 m³/s. Vista del codo, zona de aireación, transición y túnel.

En la parte inferior del codo se han registrado presiones positivas para los valores medios en todas las tomas, alcanzando en algunas de ellas valores máximos que superan los 25 m.c.a. y valores mínimos negativos entorno a los 5 m.c.a.

Los gráficos de los registros efectuados en las tomas de la parte superior del codo demuestran que en la zona baja, próxima a la salida entre las tomas nos 10 a 30, las presiones tienen valores medios negativos, incluso en algunas tomas los valores máximos también son depresiones. Se han anotado valores muy por debajo del cero absoluto. En las tomas situadas en las zonas más altas, las presiones son positivas en la mayoría de los casos incluso para los valores mínimos, aunque en algunos puntos se han registrado valores mínimos negativos por debajo de 10 m.c.a.

3.4 *Resultados*

El funcionamiento de la solución proyecto se ha comprobado ineficaz para prácticamente todos los caudales, solo se podria admitir el funcionamiento para caudales inferiores a 600 m³/s el cual esta muy lejos del caudal de proyecto, 1.125 m³/s.

Aunque la capacidad de desagüe es suficiente para conseguir que pase la avenida extrema, las depresiones registradas en la parte superior del codo hacen que la solución no sea viable.

En la Figura 8 se muestra un gráfico de una de las tomas de la parte inferior del codo en la que se ve que, si bien la presiones medias son positivas, existen depresiones importantes. Se puede observar como las presiones se disparan y se alejan de la atmosférica en torna al caudal de 600 m³/s.

En la siguiente figura, se muestra un gráfico de una de las tomas de la parte superior del codo en la

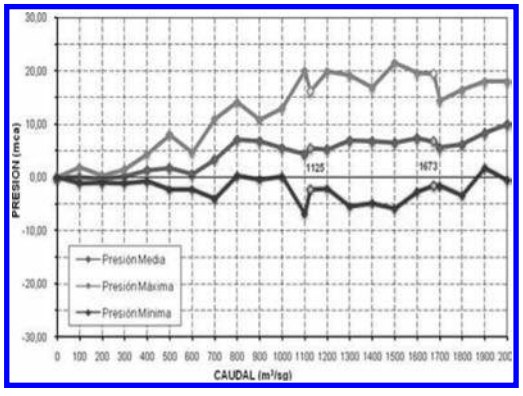

Figura 8. Grafico de una toma de presión en la parte inferior del codo.

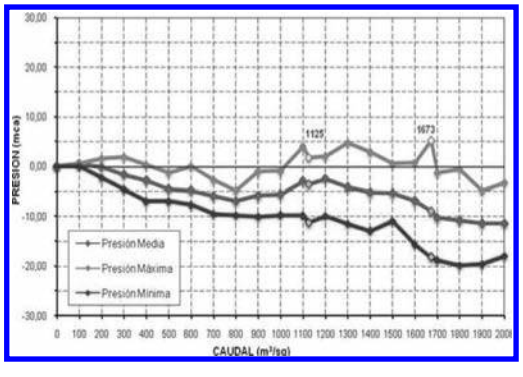

Figura 9. Grafico de una toma de presión en la parte superior del codo.

que se puede observar las grandes depresiones que se crean para prácticamente todos los caudales.

Se realizaron algunas modificaciones a la solución proyecto como la colocación de pilas antivortice y la reducción de la sección de salida.

En cuanto a las pilas, si bien mejoraban la alimentación y eliminaban el remolino, no sirvieron para guiar el flujo y eliminar el penacho central. En cuanto a las depresiones las pilas no tuvieron ningún efecto.

Otra de las modificaciones realizada fue disminuir la sección de salida del codo. Teniendo en cuenta que la capacidad de desagüe era más que suficiente, se redujo la sección de manera que se provocara el anegamiento parcial del aliviadero. Esta solución tampoco mejoro las depresiones en la parte superior del codo.

4 ALTERNATIVAS ENSAYADAS

Paralelamente a los trabajos en modelo físico se esta llevando a cabo un estudio en modelo matemático del aliviadero de Calanda al objeto de analizar diferentes soluciones que permitan corregir las anomalías de funcionamiento detectadas en el modelo físico. Con este estudio se pretende minimizar los futuros ensayos de las alternativas en modelo físico.

El estudio se divide en dos fases: la primera de calibración del modelo matemático comparando con los resultados obtenidos en el modelo físico, y la segunda relativa al análisis de modificaciones o de nuevas soluciones.

4.1 *El modelo matemático SPH*

Para la realización de este estudio se ha empleado el modelo MDST desarrollado en el CEDEX. Se trata de un modelo Lagrangiano de partículas SPH que resuelve las ecuaciones de Navier Stokes en 3D para fluido cuasi-compresible.

Para la discretización del medio fluido se han empleado 1.500.000 partículas de 40 cm.

En el modelo SPH se ha caracterizado correctamente el funcionamiento hidrodinámico del aliviadero, y aunque existan ciertas discrepancias en los valores de las presiones en ambos modelos, achacables a los efectos de inercia y de aireación, ambos modelos coinciden en detectar un funcionamiento anómalo en la fase de puesta en carga del codo en el entorno de los caudales de la avenida de proyecto (800–1300 m^3/s), puesto que se aprecia que cuando el pozo no está anegado existe una separación de la lámina en el contorno interior del pozo.

4.1.1 *Aliviadero frontal curvo*

Una de las alternativas ensayadas con el modelo SPH consistió en desarrollar la sección circular de la embocadura de la solución proyecto.

Esta solución consiste en un vertedero curvo semicircular enfrentado al túnel y una rápida convergente con la que se pretende conseguir un correcto guiado del flujo mediante la ayuda de los cajeros, para conducirla al túnel. Para evitar la existencia de ondas en el túnel se genera un estrechamiento en la embocadura que peina las ondas.

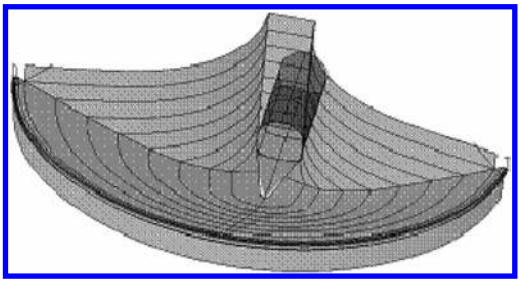

Figura 10. Aliviadero frontal curvo.

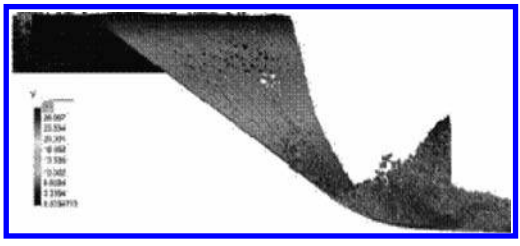

Figura 11. Funcionamiento para la avenida de proyecto.

En esta configuración se rebajó a sección de entrada al túnel de manera que se provocaba la puesta en carga a la entrada y se conseguía una circulación sin perturbaciones por el túnel.

En la siguiente figura se muestra el funcionamiento de esta solución para el caudal de la avenida de proyecto. Se aprecia un buen comportamiento en general, aunque requiere algunas mejoras para evitar los despegues laterales de los cajeros y reducir el penacho que se forma a la entrada al túnel.

4.1.2 *Sifón invertido*

Otra de las alternativas ensayadas en el modelo matemático fue la denominada sifón invertido.

Esta solución partía de la embocadura de la solución proyecto, luego pasaba a un pozo vertical y este conectaba con el túnel mediante la unión de dos codos uno de 180º que sería la parte baja del sifón y otro de 90º.

Con este diseño se ha pretendido garantizar la puesta en carga de la embocadura al túnel para toda la gama de caudales, manteniendo la misma embocadura que en la solución proyecto.

El funcionamiento es correcto para toda la gama de caudales ensayados, pues la carga hidrostática de presiones evita la existencia de depresiones en el interior de la curva.

Esta solución, si bien eliminaba todas las depresiones que se generaban en la solución proyecto, provocaba unas sobreexcavaciones que no eran admisibles.

4.2 *Modelo fisico*

Utilizando parte del modelo físico construido para ensayar la solución proyecto se realizaron dos alternativas. La primera de ellas consistió en reducir la sección de entrada a un tercio aproximadamente de la sección de la solución proyecto y disponer un laberinto para conseguir la longitud de vertido necesaria. La otra alternativa consistió en acoplar un prisma hexagonal a modo de pozo y unir este con el túnel a través de una embocadura de sección rectangular.

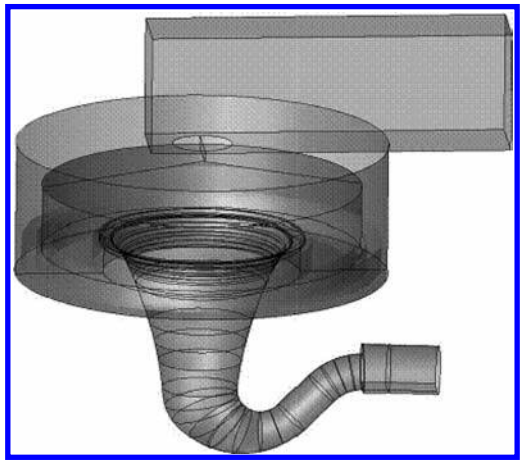

Figura 12. Sifón invertido.

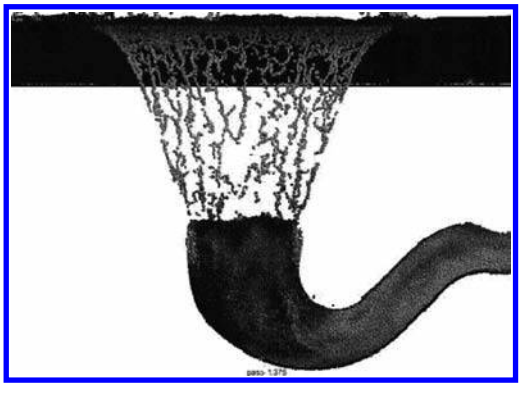

Figura 13. Funcionamiento del sifón invertido.

4.2.1 *Laberinto con salida en túnel*

Para la realización de este ensayo se utilizo un laberinto ensayado anteriormente en el CEH. Este laberinto se instalo sobre el modelo de la solución proyecto de forma que se aprovechaba parte de la entrada, el pozo y el túnel, tal y como se puede ver en la siguiente fotografía. El laberinto, de planta curva y 5 módulos, desarrolla una longitud de 169,5 m. Con esta configuración se consiguió desaguar al avenida extrema funcionando en todo momento en régimen libre. Sin embargo los niveles alcanzados sobrepasaban la coronación de la presa. Esta tipología es la que se esta estudiando actualmente y se desarrolla en el apartado 5 de este articulo.

4.2.2 *Pozo*

La última de las alternativas estudiadas consistió en la construcción de un prisma hexagonal de 15,15 m de lado y 27 m de altura. Este prisma

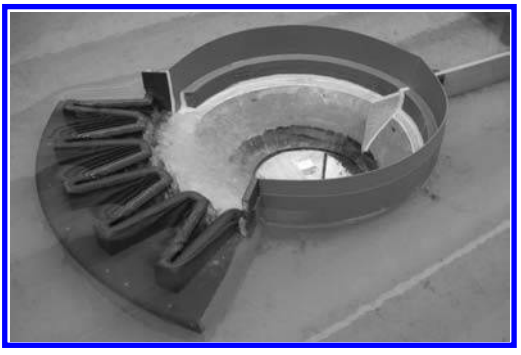

Figura 14. Laberinto con 5 módulos instalado sobre la solución proyecto.

Figura 15. Alternativa en pozo.

se acoplo a una sección intermedia de pozo de la solución proyecto. La unión entre este pozo y el túnel se realizó mediante una pirámide truncada de base una sección rectangular de 97 m^2, correspondiente a la salida del pozo, y la parte superior tenia una sección rectangular de 77 m^2.

El funcionamiento del pozo era correcto tanto para caudales pequeños, cuando funcionaba en régimen libre, como para los caudales mayores para los que funcionaba como un desagüe bajo compuerta, con el túnel siempre en régimen libre.

Esta solución fue descartada por los técnicos de la Confederación por problemas tanto de mantenimiento como por el volumen de excavación que generaba.

5 SOLUCIÓN ACTUAL

Entre las distintas soluciones estudiadas, se consensuó con la Confederación del Ebro escoger aquella que permitia un desagüe en régimen libre para toda la gama de caudales de ensayo, evitándose asi efectos difícilmente controlables que se pueden presentar en otras soluciones. En este sentido la solución actual se basa en la alternativa 4.2.1, en esta alternativa, si bien la altura de la lamina de agua superaba la coronación de la presa, se comprobó que el túnel era capaz de desaguar la avenida extrema sin entrar en carga. Basándose en esta solución se definió un nuevo laberinto que, con mayor longitud de vertido, fuera capaz de desaguar la avenida extrema manteniendo un resguardo suficiente.

Para ello se fijó una altura de lámina de agua h = 3 m, limite para la avenida extrema, y se mantuvo el calado de aproximación al umbral del aliviadero p = 10 m (diferencia entre la plataforma 430 y la cota 440 umbral del aliviadero).

Con estas premisas se comprobó que la relación h/p es menor que 0.5, condición necesaria para el correcto funcionamiento del aliviadero en laberinto. Con estos datos se definió un modulo de laberinto con las siguientes características:

l = 46,71 m
w = 11,11 m
a = 1,5 m
b = 20,84 m

La longitud total necesaria para poder desaguar la avenida extrema fue de 243 m, siendo necesario colocar 5 módulos.

Estos módulos se disponen en planta sobre un arco de circunferencia de Radio 52 m.

Para pasar del laberinto al túnel se utiliza, en primer lugar un pequeño cuenco situado a la cota 425. Este cuenco queda definido en planta por el arco de circunferencia del aliviadero, los cajeros que dirigen el agua hacia la rápida y el umbral de dicha rápida. Tiene una longitud de 17,25 m y una profundidad de 2,5 m, medido desde el umbral de la rápida.

La rápida esta definida en planta por una recta de 40 m de longitud y dos cajeros convergentes, estos están formados por dos alineaciones curvas tangentes entre si y tangente a los cajeros del aliviadero y del túnel.

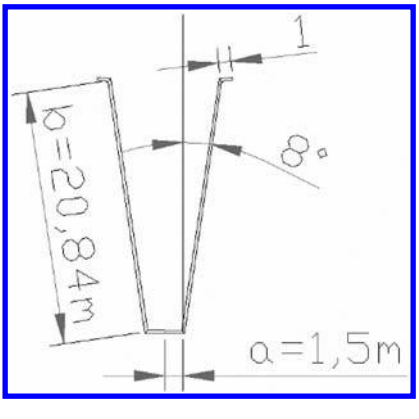

Figura 16. Definición de un módulo de laberinto.

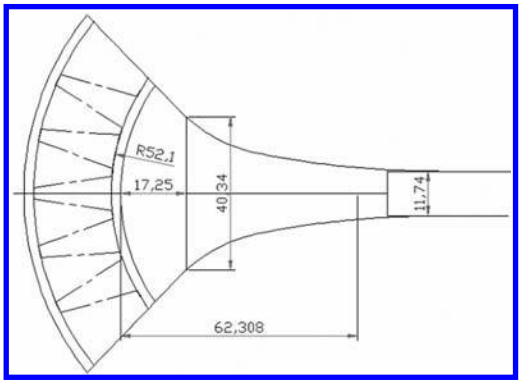

Figura 17. Planta del laberinto y la rápida.

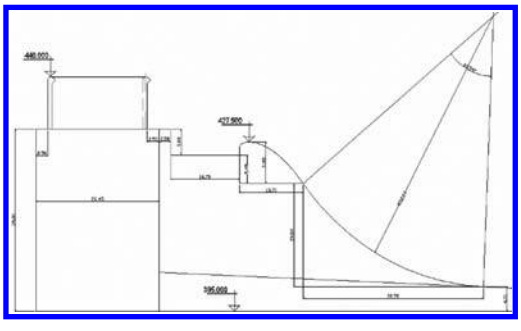

Figura 18. Alzado del laberinto y la rápida.

Figura 19. Modelo escala 1/60.

En alzado esta definida por un perfil tipo Creager para h = 7 m y un arco de circunferencia de radio 52.04 m que es tangente al perfil Creager a la cota 419.55 y a la solera del túnel.

El umbral de vertido de la rápida se sitúa a la cota 427,5.

Este laberinto se ensayó reproduciendo tanto el laberinto y la rápida como un tramo de túnel, todo ello a escala 1/60 para poder ensayar en uno de los canales de que dispone el CEH.

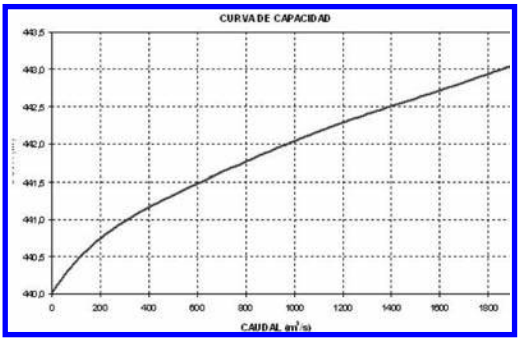

Figura 20. Curva de capacidad del laberinto.

Figura 21. Funcionamiento del aliviadero para la avenida de proyecto.

El laberinto se realizo en PVC transparente, la rápida en cemento blanco.

El resto de los elementos se realizaron con materiales fácilmente modificables, como yeso o madera, de forma que se pudieran cambiar rápidamente hasta conseguir la solución optima.

Los ensayos realizados se dirigieron a la obtención de la curva de capacidad y a las modificaciones necesarias para definir correctamente la rápida.

Como puede observarse en la curva de capacidad siguiente, se consigue desaguar la avenida extrema con una altura de lámina de agua de 2,9 m, cumpliendo con las exigencia del proyecto.

También se realizaron medidas de altura de agua en el túnel, comprobando que existía suficiente resguardo para que siempre funcionara en régimen libre.

Como se puede apreciar en las fotografias siguientes el funcionamiento del aliviadero, para la avenida extrema Q = 1.673 m^3/s, es bastante bueno ya que no se produce grandes interferencias entre las laminas vertientes y tampoco se alcanzan grandes alturas en el túnel.

Figura 22. Funcionamiento del aliviadero para la avenida extrema.

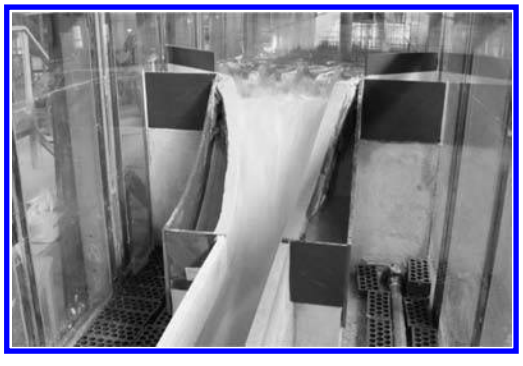

Figura 23. Vista general del aliviadero funcionando con la avenida extrema.

6 RESUMEN Y CONCLUSIONES

La solución proyecto, una vez ensayada, se descartó, debido a que, aunque tenía capacidad suficiente para desaguar la avenida extrema, las depresiones registradas en la parte superior del codo eran demasiado elevadas.

Además la configuración del aliviadero favorece la formación de un penacho, para caudales hasta 900 m^3/s, que impacta sobre la parte superior del codo.

Para caudales superiores a los 900 m^3/s se forma un remolino que si bien no afecta a los niveles en el embalse si afecta a las presiones.

De las soluciones analizadas en el modelo matemático, la primera de ellas, aliviadero frontal curvo, se descartó por dos motivos de carácter constructivo.

El primero de ellos es los problemas de excavación y sostenimiento de la estructura derivados de las características geotécnicas de la zona.

El segundo era el incremento del volumen de obra tanto en excavaciones como en hormigón.

La segunda, denominado sifón invertido se descartó por el exceso de excavación que producía y los problemas de mantenimiento derivados de la zona baja que se encontraría permanentemente inundada.

En cuanto a las alternativas ensayadas en el modelo físico, actualmente se esta ensayando la tipología definida en el apartado 4.2.1 que, como ya se ha indicado, será posiblemente, la solución sobre la que se realizará el proyecto de construcción.

La alternativa en pozo aunque funcionaba correctamente se descarto por motivos constructivos, exceso de excavación, y los problemas de mantenimiento que podría generar este tipo de estructura.

Por ultimo en lo referente a la solución actual se ha ensayado uno de los aliviaderos en laberinto funcionando siempre en régimen libre, ya que la lamina de agua para la avenida extrema queda muy por debajo de la clave del túnel.

La altura de la lámina de agua en el embalse para la avenida de proyecto y la extrema son de 2,2 m y 2,9 m respectivamente. Alturas muy similares a las de la solución proyecto.

Esta previsto continuar ensayando la solución definitiva en su conjunto, es decir, los dos aliviaderos en laberinto más el aliviadero existente.

De esta manera podremos comprobar el funcionamiento conjunto y también nos permitirá definir la obra de reincorporación de los tres aliviaderos al cauce.

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Sustitución de compuertas tipo vagón en el aliviadero de la presa de Cazalegas

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ABSTRACT: The Cazalegas dam has 7 spillway gates. These gates are wagon type. During maintenance inspection severe corrosion was detected at gates numbers 3 and 5. The reparation of those gates was refused due to the level of corrosion, so two new gates were erected in replacement. Due to the limited admissible load of the bridges over the gates, the new ones were made capable of been transported through the water in order to reach their definitive position. This paper describes how this was achieved.

1 INTRODUCCIÓN

La presa de Cazalegas, perteneciente a la Confederación Hidrográfica del Tajo es una presa de gravedad y materiales sueltos, con una altura sobre cimientos de 16 m y una longitud de 1004 m. Su capacidad es de 7 hm^3. Suministra agua para el abastecimiento a Talavera de la Reina y otros municipios próximos, y para la Zona Regable del Alberche, de unas 10.000 ha.

La construcción de la presa se finalizó en el año 1949. Dispone de un aliviadero formado por siete vanos cada uno de ellos con compuertas tipo vagón de 11.000 × 5.000 mm.

Durante los trabajos de mantenimiento que se realizaban en el año 2008, se detectó, al retirar la pintura en dos de las compuertas de aliviadero, un importante grado de oxidación en elementos estructurales del tablero de la compuerta que aconsejaron su sustitución.

Se tenía el handicap de que la capacidad de carga del puente en el aliviadero es muy limitada por lo que no era recomendable ni siquiera transitar por el mismo con grúas y ni siquiera con un camión cargado con el propio tablero.

Este grave inconveniente unido a que la presa de Cazalegas, debido a sus servidumbres de explotación, siempre mantiene el nivel de agua tocando las compuertas, complicaba la operación de sustitución de los tableros.

Se optó por la opción de colocar los tableros transportándolos por el propio agua del embalse.

Se realizaron los tableros de forma que, cumpliendo su misión final de obturación, fueran flotantes, de forma que se pudieran descargar en la orilla en un punto accesible y remolcados mediante una barca hasta el punto de colocación.

Para poder retirar el tablero viejo y no verter agua se habían de colocar las ataguías disponibles en la presa pero entonces, no se podría colocar el tablero nuevo.

Se decidió entonces dotar a los tableros de flotadores adicionales que permitieran posicionarlos en posición vertical, de forma que se situaran entre el tablero antiguo y las ataguías.

El izado de los tableros nuevos se realizó con ayuda del mecanismo de accionamiento existente.

2 PROBLEMÁTICA

En el año 2.008 durante la revisión realizada en el mantenimiento preventivo de la presa se observó que las compuertas números 3 y 5 se encontraban en muy mal estado. Sobre todo en las zonas inferiores de las compuertas se observaba un avanzado deterioro por corrosión que afectaba ya a los perfiles principales de la estructura del tablero de la compuerta.

La diferencia entre estas dos compuertas y el resto se debía a que todas las fugas que se habían estado produciendo durante años en las clapetas para limpieza superficial del embalse, habían estado mojando la estructura metálica de la compuerta, alcanzando un estado de corrosión que comprometía la resistencia de las mismas.

Aparte de lo anterior, se producían acodalamientos en su movimiento de elevación.

Ante la magnitud de la reparación necesaria, se optó por la sustitución de los dos tableros por otros de nuevo diseño sin clapeta y preparados para que posibles vertidos superiores no incidieran sobre la estructura de la compuerta.

La sustitución de las compuertas debía realizarse sin vaciar ni siquiera bajar el nivel del embalse dado que de él depende el abastecimiento a Talavera de la Reina.

El problema que presentaba la sustitución de las compuertas sin necesidad de vaciar el embalse era la

falta de accesos, ya que la carretera de coronación y concretamente los tableros de los puentes de los pasos de compuertas no soportan el peso de la grúa, ni de los camiones necesarios. La posibilidad de actuar desde un estribo de la presa era imposible desde la margen derecha por la anchura de la coronación y aunque teóricamente posible desde la margen izquierda, obligaba a utilizar grúas de muy alto tonelaje dada la distancia a las compuertas y las dimensiones de las grúas necesarias.

Por la zona de aguas abajo tampoco había posibilidad de acceder con grúas sin realizar una muy costosa obra de acceso que además condicionaría el alivio de caudales.

Sin más opciones razonables sólo quedaba la solución de acceso por aguas arriba utilizando los medios necesarios para transportar los tableros por el agua.

3 REALIZACIÓN

Para llevar a cabo los trabajos se proyectó una estructura de compuerta mediante viga cajón, con el fin de conseguir flotabilidad, dotada de vertedero superior con perfil Creager para que los vertidos no cayesen sobre la estructura metálica.

Se dotó al tablero un testero lateral desmontable que, permitiese colocar el tablero en el hueco existente, colocando posteriormente el testero para completar el ancho necesario.

Resuelto el problema de la compuerta y su construcción en fábrica, se procedió al transporte a obra mediante transporte especial por sus grandes dimensiones.

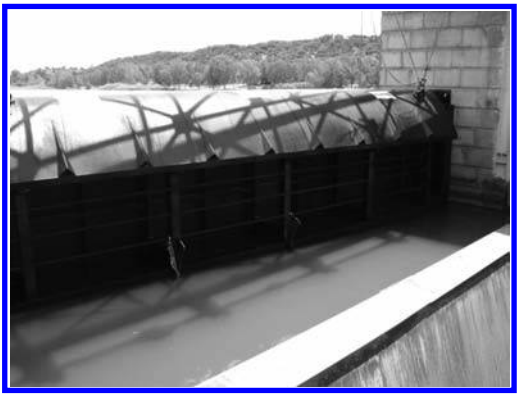
La descarga de la compuerta se realizó en el muro de coronación, donde se situó una grúa de 200 Tn (téngase en cuenta la grúa que hubiera sido necesaria para emplazarla en su hueco a unos 40 m de distancia si para descarga de camión se necesitaron 200 Tn) en la zona interior de la carretera de coronación.

El camión de transporte entra y sale por el mismo estribo dado que no se puede cruzar la zona de compuertas puesto que los puentes no soportan la carga.

Una vez descargada la compuerta y apoyada en vertical sobre la carretera se montan unos flotadores auxiliares para poder conseguir la flotabilidad necesaria, situándolos tanto aguas arriba como aguas abajo. Además de conseguir la flotabilidad el problema era que la compuerta debía poderse situar en posición vertical al llegar a su emplazamiento final, por lo que era necesario que los flotadores a su vez ayudaran a darle la inclinación deseada llenándolos o vaciándolos de agua. Los flotadores van equipados con unas válvulas manuales para la regulación del llenado y de unas boquillas de enchufe rápido para conectarlas a botellas de aire comprimido, que permiten vaciarlos de agua inyectando aire a presión que desaloja el agua. Para ayudar a conseguir esta verticalidad se colocaron también lastres de hormigón en la zona inferior de la compuerta.

A continuación con la grúa se depositó el tablero en el agua del embalse donde quedó flotando. Con una simple barca a motor se procedió a remolcarlo hasta su emplazamiento.

En general no es necesario que estas maniobras se realicen desde la coronación de la presa, pudiéndose realizar en cualquier punto que sea cómodo del embalse. En el caso de la Presa de Cazalegas se buscó el punto de más fácil acceso para los vehículos y el que exigiera menos distancia de remolque.



Ya en su emplazamiento se procedió a colocarlo posición vertical mediante el llenado de los flotadores de aguas abajo lo necesario para que, muy lentamente, se vaya inclinando hasta alcanzar la posición deseada.

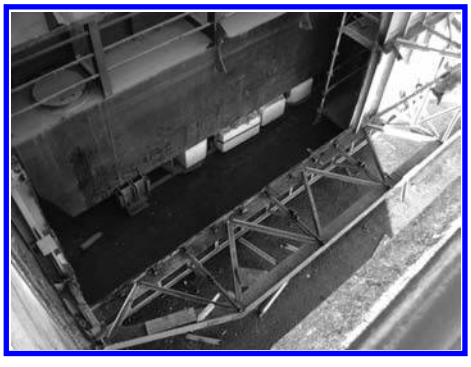
Ya vertical, se fue acercando a la compuerta a sustituir hasta dejarlo situado en el vano que existe entre la compuerta antigua y la zona de ataguías, sin que moleste a estas, amarrándolo al puente metálico existente para mantenerlo en su posición.

La colocación de las ataguías modulares existentes se realizó con el carretón existente para tal fin y que circula por la zona alta del puente metálico, en todo el recorrido de las 7 compuertas. Una vez colocadas las ataguías se abrió la compuerta antigua para vaciar la cámara existente entre ataguías y compuerta a sustituir dejándola en seco.

Posteriormente se procedió al retacado de los módulos de ataguía mediante un equipo de submarinistas hasta conseguir unas fugas admisibles para trabajar en la ubicación de la nueva compuerta.

En esta situación, además del amarre del nuevo tablero al puente metálico, se calzó la compuerta con puntales metálicos para dejarla fija en seco.

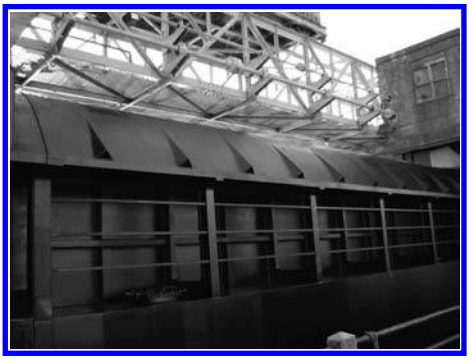
A continuación se procedió al desmontaje de la compuerta antigua troceándola en partes que pudieran ser extraídas desde el puente de coronación, procediéndose a la retira de los mismos retirada por un gestor autorizado de residuos.



Desmontada la compuerta antigua se procedió a reparar las guías, los hierros fijos y la solera inferior.

Para posicionar el nuevo tablero en su emplazamiento definitivo se enganchó con las cadenas Galle del mecanismo de elevación de la compuerta que ayudado por gatos hidráulicos estratégicamente situados permitió realizar pequeños desplazamientos en la dirección del nuevo emplazamiento. Mediante la repetición de esta maniobra, recolocando los gatos hidráulicos cada vez el tablero quedó en posición.

A continuación se revisaron los tiros de cadena Galle para que garantizar que tuvieran la misma longitud en ambas y se realizó el centrado mediante los rodillos guías, colocando a continuación los cierres de elastómero tanto laterales como de solera.



Finalmente se probó el mecanismo de accionamiento y el despegue y la elevación de la compuerta, para a continuación comprobar el funcionamiento del tablero con agua antes de retirar las ataguías y al darlo por correcto, retirar las mismas.

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Rehabilitación del desagüe intermedio de la presa de Tous (Valencia)

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RESUMEN: La presa de Tous constituye una pieza fundamental para la defensa de las avenidas del río Júcar, sirviendo además como aprovechamiento último para la regulación, distribución de riegos y abastecimiento de Valencia y Sagunto. Dentro de sus elementos de regulación, el desagüe intermedio, con capacidad para 1.200 m^3/s, es una pieza clave tanto para la regulación de avenidas como la suelta de caudales. Los conductos de descarga del desagüe intermedio se encontraban deteriorados por lo que se proyectó su blindaje mediante chapas metálicas de 15 mm de espesor para impedir el progresivo daño en los mismos. La solución adoptada mediante dos carros plataforma desde los cuales se realizan todas las actividades, transporte de chapas, posicionamiento y colocación, anclaje, soldadura, chorreados, pintura y ensayos de control de calidad ha permitido la construcción del blindaje de los conductos del desagüe intermedio de la presa de Tous de manera eficiente y segura.

ABSTRACT: The Tous Dam is a key to the defense of the Júcar river avenues, serving in adition, as the last utilization for the regulation, irrigation distribution and supply of Valencia and Sagunto. One of its elements of regulation is the intermediate drain with capacity is 1200 m^3/s, it is a key for both flood control and the release of high flows. Exhaust ducts downstream drainage were damaged by what its shielding was projected by metallic sheets of 15 mm thick to prevent the progressive damage of components. The solution adopted by two working cars from which all activities were realized, transport of plates, positioning and placement, anchorage, weld, blasted, painting and test of quality control has allowed the construction of the shielding of the intermediate drain channel of Tous's dam in an efficient and safety way.

1 DESCRIPCIÓN DE LA PRESA

1.1 *Breve reseña histórica*

Las cuencas media y baja del río Júcar reúnen un conjunto de condiciones climáticas y geográficas que favorecen la aparición de la situación meteorológica conocida como la gota fría, la cual puede originar precipitaciones de una intensidad y duración excepcionales. Como es bien conocido, una situación de ese tipo dio lugar, en Octubre de 1982, a una avenida extraordinaria que sobrepasó la coronación de la presa y produjo la erosión y destrucción de gran parte de la zona central, de materiales sueltos.

Las actuaciones necesarias para solventar, de manera global y progresiva, las avenidas catastróficas, como la última, ya referida de Octubre de 1982, así como los problemas de inundación en las cuencas media y baja del Júcar, han sido recogidas en el PLAN GENERAL DE DEFENSA CONTRA AVENIDAS DEL JÚCAR, de Junio de 1985.

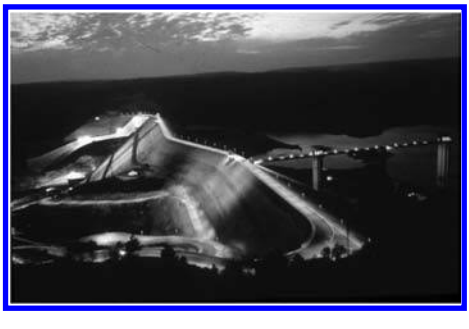

Figura 1. Vista general de la presa de Tous en la actualidad.

En dicho Plan se contempla, como uno de los objetivos fundamentales, las actuaciones a tener en cuenta para la laminación de avenidas, consiguiendo, por tanto, disminuir los caudales punta circulantes. En base a ello se llegó a la conclusión que debían construirse tres presas:

- Una sobre el río Escalona, para laminación de avenidas, con una capacidad de alrededor de 100 hm^3.
- Otras sobre el río Júcar, cerca del emplazamiento de la antigua presa de Tous, con un volumen para laminación de alrededor de 450 hm^3. Esta presa se considera pieza básica y es en la que se fundamenta el Plan.
- Y por último, otra sobre el río Albaida (presa de Bellús), con capacidad de embalse de 60 hm^3.

La presa de Tous, además de su principal objetivo, LAMINACIÓN DE AVENIDAS, cumple otros objetivos como:

- Garantía de suministro para el abastecimiento de Valencia y área metropolitana (el 75% del volumen necesario para 1,5 millones de habitantes) y el abastecimiento a Sagunto.
- Riegos de la comarca de la Ribera y canal Júcar-Turia (52.000 Ha).
- Posibilidad de producción de energía eléctrica mediante la construcción de dos centrales eléctricas.

La tipología de la presa es de materiales sueltos con un núcleo de arcilla y espaldones de escollera. Tiene una longitud en coronación de 1.024 m y una anchura de 9 m en la coronación, con una altura de 135,50 sobre cimientos. El núcleo de arcilla corona a la cota 162,00 con una anchura de 3 m, teniendo la a la cota 50,00 una anchura de 57,70 m.

El paramento de aguas arriba de la presa tiene un talud de 1,85:1,0, con berma de 9,25 m de anchura a la cota 100,00. Aguas abajo, el talud es 1,5:1,0 con tres bermas de 5,0 m de anchura, situadas a las cotas 70,100 y 130.

Los órganos de desagüe de la presa de Tous son:

- Aliviadero: Labio fijo sin compuertas. Ancho 75 m.
  Capacidad máxima de desagüe 19.700 m^3/s
  Proyectado para poder evacuar la PMF
- Desagüe intermedio: Cuatro conductos.
  Capacidad máxima de desagüe 1.200 m^3/s
- Toma de agua: Capacidad máxima de la toma 120 m^3/s
- Desagüe de fondo: Capacidad máxima de desagüe 370 m^3/s

1.2 *Desagüe intermedio*

El desagüe intermedio de la presa de Tous se sitúa sobre la estructura del aliviadero de la presa antigua, de la que se aprovecha en gran parte su cimentación.

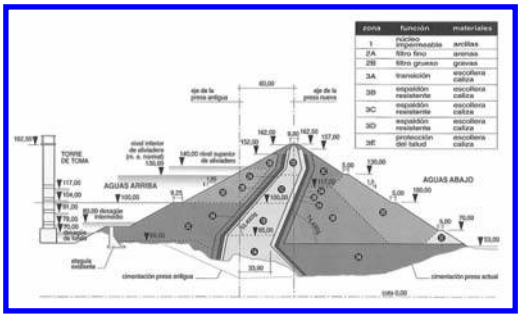

Figura 3. Planta de la presa.

Figura 2. Riadas históricas en la cuenca baja del Júcar.

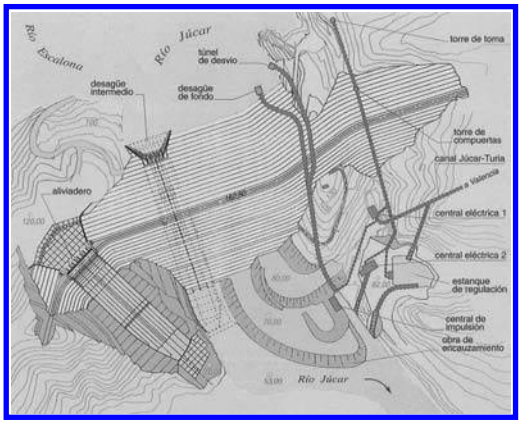

Figura 4. Planta de la presa.

El desagüe intermedio tiene las siguientes finalidades:

- Permitir un control efectivo de los niveles de agua en el embalse con cualquier caudal entrando en el mismo, fuera de los producidos en las avenidas extraordinarias.
- Proporcionar una laminación más eficaz de las avenidas.
- Satisfacer las demandas de riego de mayor entidad (P ej, el llenado del arrozal de aprox 50 m^3/s).

Figura 5. Vista general de la presa con posterioridad a la riada de 1982.

Figura 6. Desagüe intermedio durante la construcción de la presa.

El desagüe intermedio consta de tres tramos:

a. Embocadura y conductos de carga
b. Cámara de compuertas
c. Conductos de descarga.

La embocadura de 36,6 m de longitud tiene una forma hidrodinámica con altura interior variando en los 20 m a la entrada y 4 m en la conexión con los conductos de carga.

Los cuatro conductos de carga, con una longitud de 48,3 m, son de sección rectangular de 5×4 m.

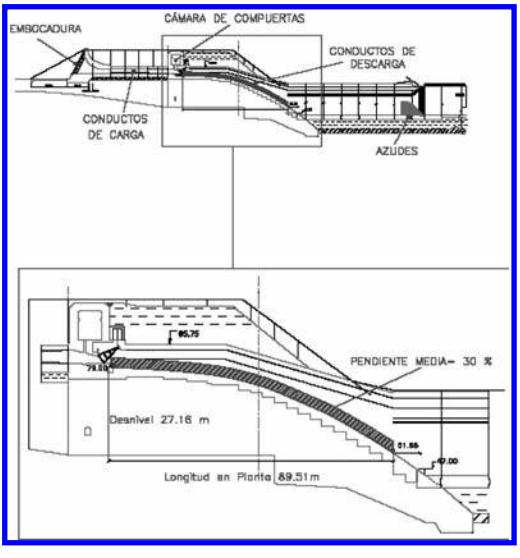

Figura 7. Perfil longitudinal del desagüe intermedio.

Figura 8. Cámara de válvulas del desagüe intermedio.

La cámara de compuertas se apoyó en el macizo de hormigón del aliviadero de la presa antigua y en ella se han dispuesto los cierres de control y seguridad del desagüe. El cierre del control está formado por 4 compuertas tipo Taintor, de 3,50 m × 3 m. Los cierres de seguridad están formados por compuertas deslizantes, tipo Bureau, de 3,50 m × 3,50 m de sección.

Los conductos de descarga, que son el objeto del blindaje que posteriormente se detallará, están situados igualmente sobre la estructura de hormigón del aliviadero antiguo. El primer tramo de 84,2 m de longitud, los cuatro conductos tiene 5 m de anchura y altura variable. En el tramo siguiente y hasta su salida al exterior, las pilas que separan los conductos de cada bloque desaparecen, quedando el desagüe a dos únicos conductos de 11,50 m de anchura y 24 m de altura entre solera y clave, formando una estructura monolítica muy protegida, con espesores de 3 m en hastiales y de 2 m en bóvedas, fuertemente armadas debido a las importantes solicitaciones que gravitan sobre la misma.

El desagüe intermedio de la presa puede ser considerado el centro neurálgico de la presa y tiene como principal cometido el control del embalse durante su explotación, pues permite desaguar 1.200 m^3/s antes de que empiece a descargar el aliviadero de labio fijo. Como ya se ha comentado en la explotación ordinaria de la presa los caudales elevados solicitados por las demandas agrarias son suministrados desde este desagüe.

2 INTRODUCCIÓN AL PROBLEMA

2.1 *Deterioro de los conductos de descarga del desagüe intermedio y geometría existente*

El deterioro de los conductos de descarga del desagüe intermedio se manifestaba con el despegue en algunas zonas del hormigón de 2ª fase, utilizado para regularizar la superficie de los conductos.

Estas pequeñas cavidades que se formaban en solera y paredes provocaban pequeños despegues de la lámina de agua con las consiguientes cavitaciones. El deterioro progresivo de los conductos hizo que se incluyese dentro de las obras de terminación de la presa de Tous una actuación que solventará dicho problema.

Geométricamente los conductos de descarga objeto de la actuación son cuatro galerías paralelas de 5 m de ancho cada una que inician desde la cota de solera 79 y van hasta la cota 45 (34% de pendiente media) que es la solera del conducto de salida y del posterior cuenco de amortiguación.

Este desnivel se recorre en aproximadamente 100 m, a partir de compuertas, siendo objeto de blindaje los 95 m primeros, que es la zona en la que

Figura 9. Deterioro de los conductos de salida del desagüe intermedio.

dos de las galerías confluyen en un único conducto de salida o "catedral".

Este desnivel junto con las dimensiones de los conductos es el principal hándicap a superar para llevar a cabo el blindaje.

2.2 *Blindaje metálico*

La solución diseñada ha sido un blindaje mediante chapas metálicas de acero sujetas con anclajes químicos M20 a la solera y laterales de las galerías.

Las chapas metálicas son de acero S-275 y e = 15 mm en toda la sección mojada de los conductos, con módulos de avance de 2,5 m mediante chapas en solera de 5 × 2,5 y alzados de 3 × 2,5.

Figura 10. Detalle de las chapas del blindaje del desagüe intermedio.

2.3 *Dificultades de ejecución*

Las dificultades de ejecución del blindaje radican en la pronunciada pendiente de los conductos en el tramo de aguas abajo, en la zona de entronque con la llamada "catedral" y la nula accesibilidad desde la parte de aguas arriba, donde la geometría del conducto permitiría una ejecución más sencilla. Debido a las grandes pendientes longitudinales y alturas existentes en estas galerías, se hacen prácticamente inaccesibles desde el inicio o fin de las mismas con medios convencionales tipo grúas, puentes grúa, andamios, polipastos, etc, e imposible el trabajo de manipulación de chapas metálicas con seguridad y precisión requerida para la colocación de las mismas, dado que el peso de cada chapa es de unas 1,8t.

Por otra parte, los trabajos debían ejecutarse en un plazo razonable pues existían otras actividades relacionadas con el desagüe intermedio que no podían comenzarse hasta que hubiese finalizado el blindaje, como eran los azudes al final de los conductos de descarga.

La exigencia en cuanto a calidad de ejecución, seguridad y plazo debían ser satisfechas por el método constructivo seleccionado.

3 SOLUCIÓN ADOPTADA. PROCEDIMIENTO CONSTRUCTIVO

Se aborda la ejecución de las cuatro galerías instalando en cada uno de los dos conductos de salida (en la parte final horizontal) talleres de mecanizado, soldadura y pintura, asistidos por puentes grúas para realizar el trasiego de material entre cada taller, en zona de acopio y carga a los carros.

Se estudian distintos procedimientos para la ejecución y ninguno permite la realización integral de la misma, siendo todas incompletas, arriesgadas, e inseguras. Por ello se desestiman.

La solución final adoptada es la de dos carros polifuncionales "ad hoc", que permiten su uso como plataforma de transporte de materiales, útil de posicionamiento y colocación de las chapas y plataforma de trabajo para todas las actividades de anclaje, soldadura, chorreados, pintura, y ensayos de control con unas medidas excepcionales de seguridad para los operarios, y permite una zona de trabajo limpia, sin cables eléctricos, mangueras de equipos de soldadura etc.

El procedimiento de ejecución del blindaje, a grandes rasgos es el siguiente:

1. Preparación en taller especializado de las planchas con el corte a medida, taladrado, granallado e imprimación de ambas caras y preparación de bordes.
2. En talleres de obra, se colocan y sueldan los casquillos de los inyectores, las chapas de continuidad entre cajones consecutivos y las piezas posicionadoras de rasante.

Figura 11. Vista general de la zona del canal de descarga. Acopios de chapas a la derecha, taller de pintura a la izquierda.

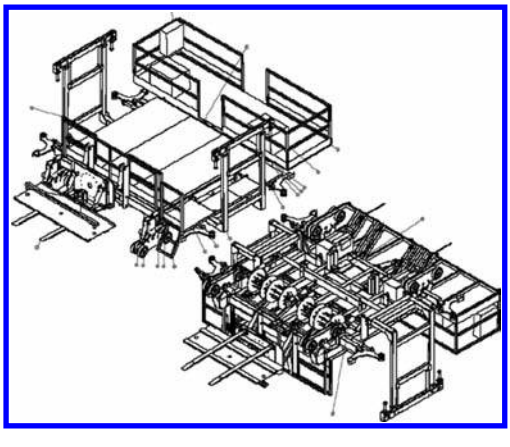

Figura 12. Despiece de las plataformas de trabajo.

3. Se chorrean las partes soldadas en obra y afectadas por la proximidad de las operaciones, y se impriman.
4. En cámara de pintado se aplican las distintas capas de pintura, comprobando tiempos de aplicación, espesores y curado de cada capa.

Figura 13. Separadores en las chapas del blindaje.

Figura 14. Pintado de las chapas mecanizadas.

Figura 15. Labores de pintado en el taller situado en obra.

5. Cada modulo (2 chapas de laterales y una chapa de fondo o solera) es cargado sobre el carro, que las transporta a su lugar de colocación plegadas y abatidas sobre la horizontal.

El carro circula sobre las chapas colocadas previamente, pero sin inyectar ni soldar, arrastrado por los cables de un tractel. Para poder circular con seguridad y evitar la deformación de las mismas, se realizan una parte de los taladros de fijación de la chapa mediante

Figura 16. Estado final de las chapas tras el pintado.

Figura 17. Premontaje de las chapas del blindaje.

Figura 18. Detalle del anclaje químico.

anclajes de altas resistencias iniciales, de manera que a las pocas horas de su colocación adquieran resistencia suficiente para poder permitir el paso del carro sin producir deformaciones.

En primer lugar se monta la chapa de fondo que queda fijada con topes y uñetas, se mueve el carro sobre la chapa y se fija esta con 4 anclajes químicos rápidos HVU-M20-170 (a los 5 minutos resistencia de 9t). El anclaje consiste en realizar la perforación, soplar, colocar el cartucho y meter la varilla roscada.

Estos primeros anclajes se realizan lo más próximo a la zona de rodadura. A los 5 minutos ya se puede realizar la soldadura de estos anclajes con la chapa, después se premontan las chapas laterales derecha e izquierda. Se ejecutan 4 anclajes químicos en cada una de las chapas, con lo que la chapa ya queda fija en su posición definitiva y el carro queda liberado para las siguientes maniobras.

También se complementa la operación con el galgado de los separadores chapa-hormigón, adaptándoles al espesor de hueco real en cada punto.

Todas estas maniobras de premontaje se realizan en el turno de día y sin ninguna actividad simultanea en la misma galería.

Colocada la chapa fija en su posición, en el siguiente turno de noche se realizan todas la perforaciones inyectando la resina (y/o cartucho) colocando las varillas. A las 8 horas ya se pueden soldar las varillas a la chapa quedando de esta forma perfectamente colocada.

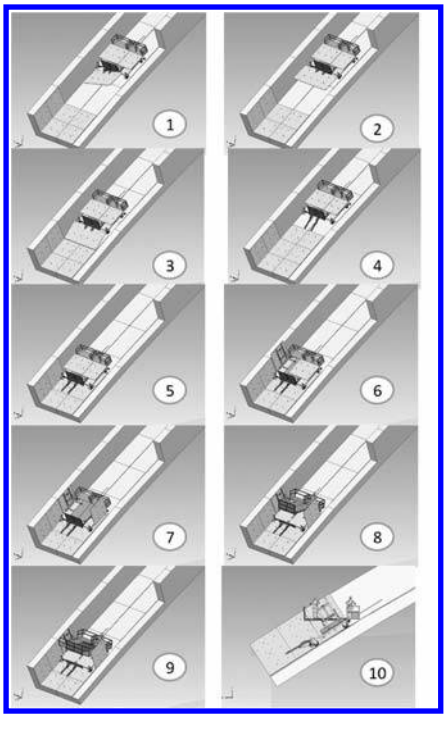

Figura 19. Descripción de los pasos de la plataforma para el montaje de un modulo.

Figura 20. Maniobras de premontaje desde la plataforma.

Figura 21. Anclajes de fijación de la chapa metálica.

Figura 22. Ejecución de anclajes en chapas laterales.

Este ciclo de carga, transporte, fijación previa y posterior bajada a recoger nuevo conjunto dura aproximadamente entre 4 y 5 horas, colocándose entre 2 y 3 módulos diarios.

Este proceso se repite hasta completar la totalidad de cada una de las galerías (38 módulos).

Una vez se han fijado la totalidad de los anclajes se procede a la soldadura completa de los mismos. A medida que se van realizando la soldadura de raíz se procede al montaje de los sectores circulares, y la soldadura de estos con la chapa, que se producen entre dos chapas consecutivas de alzado. Estas piezas están hechas a medida una vez colocadas las chapas adyacentes.

Figura 23. Trabajos de soldadura desde la plataforma.

Figura 24. Trabajos de soldadura entre modulos.

6. Las operaciones finales de montaje finalizan con todas las piezas especiales de remate de cajones con compuertas, de borde superior de cajones verticales, de cierre entre galerías frontalmente, y de la cuña inferior de finalización y acuerdo de pendientes.

Figura 25. Soldadura entre chapas.

Figura 26. Inyección del contacto chapa-hormigón.

Figura 27. Estado final de una chapa lateral tras el soldado de tapones y anclajes.

Figura 28. Trabajos de pintado del blindaje.

7. Una vez soldados los módulos completos, se realiza la inyección de contacto entre hormigón y chapa. Para ello, se utiliza un mortero de alta resistencia de 40 MPa, autonivelante y sin retracción para poder garantizar el correcto contacto chapa-hormigón. La inyección se realiza de abajo arriba, llevando dos circuitos de inyección paralelos, y comprobando el rebose de lechada con los inyectores de control dispuestos para ello.
8. Realizada la inyección se procede a la colocación y soldadura de los tapones de cierre de inyección, que previamente han estado cerrados por un tapón de material plástico.
9. Se procede a granallar e imprimar nuevamente la totalidad de la superficie con grado SA3, para proceder a la aplicación de la capa de revestimiento definitiva con 500 micras de poliuretano aromático y 80 micras de poliuretano alifático con certificado de potabilidad.

Por último aquí tenemos una foto con el blindaje terminado.

Figura 29. Blindaje del desagüe intermedio terminado.

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Estudio de la rotura de presas de material granular grueso uniforme no cohesivo sin elemento impermeable. Fase de inicio

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RESUMEN: Se presentan los resultados obtenidos mediante modelación física de la rotura de presas de escollera por sobrevertido, los trabajos se enmarcan en el proyecto de investigación del Plan Nacional de I + D 2007/2011 denominado XPRES. Los ensayos en modelo físico han servido para evaluar el papel de una serie de parámetros como la altura de presa H, el talud aguas abajo N, y el diámetro medio de la escollera D_{50}. El material utilizado en estos ensayos tiene las características de un material granular grueso uniforme no cohesivo. En este proyecto participan de forma coordinada la UPM el CEDEX y el CIMNE.

ABSTRACT: We report the results obtained by physical modeling of rockfill dam failure due to overtopping, the works was framed in the National Research Program for I + D called XPRES 2007/2011. The physical model tests were used to assess the role of a number of parameters such as dam height H, the downstream slope N, and mean diameter D_{50}. The material used in these trials was noncohesive homogeneus. This project has been coordinated between the UPM, CEDEX and CIMNE.

1 INTRODUCCIÓN

Cuando se inicia el sobrevertido en una presa de escollera, la presa es capaz de evacuar un cierto caudal sin presentar daños apreciables en el espaldón de aguas abajo. Una vez superado ese caudal se iniciará la rotura de la presa con pequeños desplazamientos a pie de presa que irán creciendo hasta llegar a la coronación de aguas abajo. En este instante consideraremos que la fase de inicio de la rotura se ha completado. Una serie de ensayos en modelo físico se han llevado a cabo en el Laboratorio de Hidráulica de la E.T.S.I. de Caminos de la UPM para estudiar el proceso de rotura de las presas de escollera y desarrollar criterios de diseño de protecciones eficaces. En esta comunicación se presentan los resultados correspondientes a los ensayos de roturas de presas sin elemento impermeable.

1.1 *Objetivos*

El objetivo de este programa de ensayos ha sido:

- Identificar los mecanismos de rotura en una presa de escollera por sobrevertido.
- Determinar los parámetros más importantes en el proceso de rotura.
- Conocer la evolución del proceso rotura.
- Conocer el caudal de inicio de la rotura.
- Conocer el caudal de rotura de la presa, entendido como aquél que, mantenido un tiempo suficiente, completa la fase de inicio de la rotura, hasta que ésta alcanza la coronación de la presa y por tanto amenaza el elemento impermeable.

La figura 1 muestra las variables principales que miden el avance de la rotura en la presa.

Donde:

- A: Avance de la pendiente inferior de la brecha desde el pie de aguas abajo.
- B: Avance total de la brecha desde el pie de aguas abajo.
- H: Altura de la presa.
- h_a: Nivel de agua aguas arriba.
- h_b: Nivel de agua aguas abajo.
- M: Talud de aguas arriba.
- N: Talud de aguas abajo.

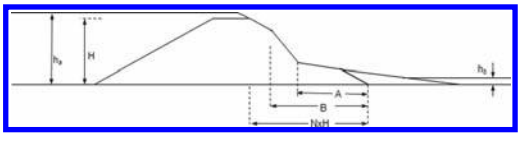

Figura 1. Esquema del avance de la rotura en la presa.

Cuadro 1. Campaña de ensayos.

Ensayo N°	D_{50} (mm)	H (m)	N
1	8,2	0,6	1,5
2	8,2	0,6	1,75
3	8,2	0,6	2,5
4	8,2	0,6	2,7
5	12,64	1,0	2,2
6	12,64	1,0	3,0
7	35	1,0	1,5
8	35	1,0	2,2
9	35	1,0	2,5
10	35	1,0	3,0

2 PLANTEAMIENTO GENERAL

Se han realizado ensayos en modelo físico de presas de escollera sin elemento impermeable, en los que se ha evaluado el comportamiento de la presa frente a una rotura por sobrevertido. El desarrollo de los ensayos se ha llevado a cabo por escalones de caudal hasta conseguir que la rotura llegue a la coronación. El incremento de caudal se ha realizado cuando la presa se encontraba nuevamente en un estado estacionario. El tiempo establecido para considerar que la presa ya no tiene más movimientos es variable dependiendo de la fase de la rotura en que se encuentre. Los tiempos son más cortos en la etapa inicial y más largos en la etapa final de la rotura. Se han realizado un total de 10 ensayos, manteniendo un talud aguas arriba $M = 1,5$; un ancho de coronación $C = 0,20$ m y un ancho de presa $W = 2,46$ m.

Los ensayos se han realizado en un canal de grandes dimensiones al que llamaremos en adelante recinto de ensayos, construido exclusivamente para este proyecto. Las dimensiones del recinto son: 2,46 m de ancho × 1,3 de alto × 13,5 m de largo.

3 INSTALACIONES Y PROCEDIMIENTO DE ENSAYO

El recinto está alimentado desde un depósito de nivel constante a través de una tubería de 350 mm, dotada de un caudalímetro de ultrasonido. En la parte final incorpora una válvula motorizada que regula el caudal de entrada al modelo. El registro de los niveles aguas arriba, aguas abajo y en el vertedero de salida se realiza con sondas de nivel de ultrasonido. El ensayo se inicia con un caudal pequeño, que se va incrementando hasta que la rotura llegue a la coronación, momento en que finaliza el ensayo.

En cada escalón se registran el caudal de entrada, los niveles aguas arriba y aguas abajo de la presa, el caudal de salida y el grado de avance de la rotura de la presa. La incorporación de instrumentación se ha realizado de forma continua.

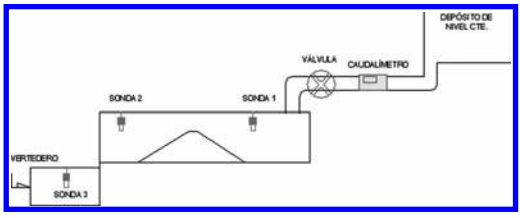

Figura 2. Esquema de las instalaciones de ensayo.

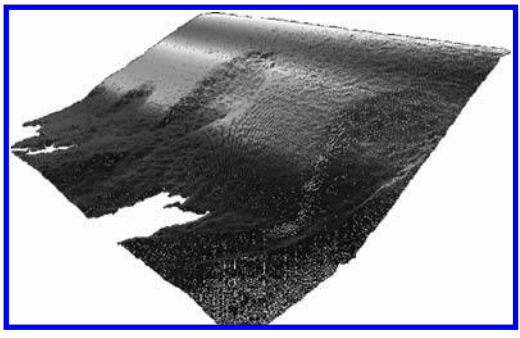

Figura 3. Modelo Digital del Terreno (MDT).

3.1 *Instrumentación*

Durante el desarrollo de los ensayos el recinto ha contado con un caudalímetro de ultrasonido, una válvula motorizada, 3 sondas de nivel de ultrasonido y un panel de 84 piezómetros para las lecturas de presión en la base de la presa. Se implementó un sistema de control a través de la captura de las señales eléctricas de los instrumentos y los ensayos son controlados desde un ordenador con software Labview.

3.2 *Fotogrametría*

La técnica de fotogrametría del objeto cercano acondicionando dos cámaras fotográficas sincronizadas, han permitido, sacando pares de fotos cada 10 segundos durante los ensayos, obtener el modelo Modelo Digital del Terreno (MDT) de la presa en cada escalón de rotura.

4 MATERIALES

Los materiales utilizados tienen gradación uniforme, dado que uno de los objetivos de este trabajo es analizar el efecto del tamaño de la partícula a través del D_{50}. Las características de los materiales utilizados se muestran en el cuadro 2.

Cuadro 2. Características de los materiales.

Descripción (mm)	Material 1 M1	Material 2 M2	Material 3 M3
D_{10}	5,97	8,36	23,68
D_{50}	8,20	12,64	35,00
D_{60}	8,70	12,88	36,91
C_u	1,46	1,54	1,56

5 RESULTADOS

Para presentar los resultados obtenidos en el laboratorio definiremos algunas conceptos:

Caudal de inicio de la rotura Q_{ir}. La presa puede evacuar un cierto caudal sin presentar cambios en su configuración inicial. Una vez superado ese caudal se iniciarán los primeros desplazamientos de parte del material del espaldón aguas abajo de la presa, lo que supone el inicio de la rotura.

Caudal de rotura Q_r. Es el mínimo caudal de sobrevertido que lleva la rotura hasta el extremo de aguas abajo de la coronación de la presa.

Caudal de rotura interpolado Q_{ri}. Como ya se comentó es complicado alcanzar el caudal de rotura exacto. Esta medida Q_{ri}, nos da una aproximación más cercana al interpolar entre el valor anterior y posterior al alcance de la coronación.

Camino de rotura *CR*. Es el camino que caracteriza el avance de la rotura desde su inicio en el pie de presa hasta llegar a la coronación. Queda definido por el gráfico que tiene en abscisas el caudal correspondiente a cada escalón y en ordenadas el grado de avance de la rotura (B).

A continuación se muestran los resultados obtenidos en estos ensayos:

5.1 *Material N°1 D_{50} = 8,2 mm*

Se realizaron pruebas para una presa de altura 0,60 m y cuatro taludes distintos aguas abajo, N = 1,5/1,75/2,5/2,7.

5.1.1 *D_{50} = 8,2 mm talud N = 1,5*

La rotura de esta presa con talud N = 1,5 está gobernada por el deslizamiento en masa del espaldón. La duración del sobrevertido es generalmente suficiente para la saturación parcial o total del cuerpo de la presa y la instalación de un campo de presiones intersticiales correspondientes que produce el deslizamiento.

5.1.2. *D_{50} = 8,2 mm talud N = 1,75*

El mecanismo de rotura para esta presa con talud N = 1,75 es un mecanismo combinado de deslizamiento en masa y arrastre de partículas, pero con claro predominio del deslizamiento en masa. Se aprecian unos ligeros arcos en la zona deslizada, que denotan la presencia del mecanismo de arrastre.

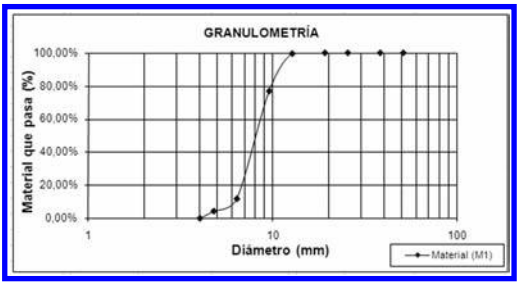

Figura 4. Material N°1.

Cuadro 3. Datos del ensayo N°1.

N = 1,5 Escalón	Q (l/s)	h_a (mm)	h_b (mm)	A (m)	B (m)
E_1	6,5	202	20	0	0
E_2	10,5	258	25	0	0,304
E_3	11,3	268	25	0,294	0,332
E_4	14,3	306	28	0,358	0,554
E_5	18,4	356	30	0,389	0,771
E_6	21,5	384	35	0,546	1,038
Q_{ri}	19,9	–	–	–	0,9

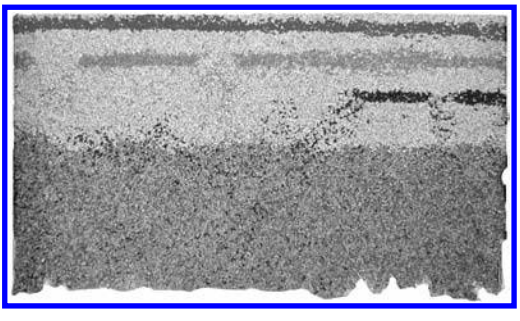

Figura 5. Foto ortogonal Q = 18,4 l/s.

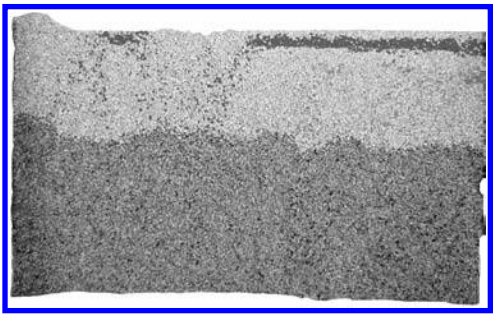

Figura 6. Foto ortogonal Q = 21,5 l/s.

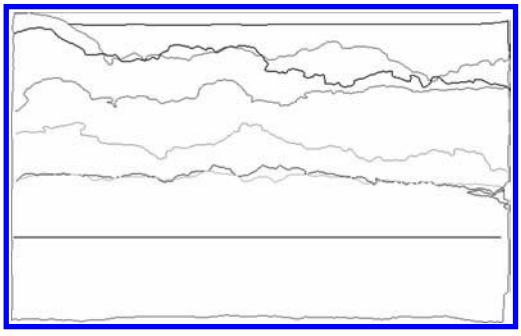

Figura 7. Avance en la rotura de la presa por escalones.

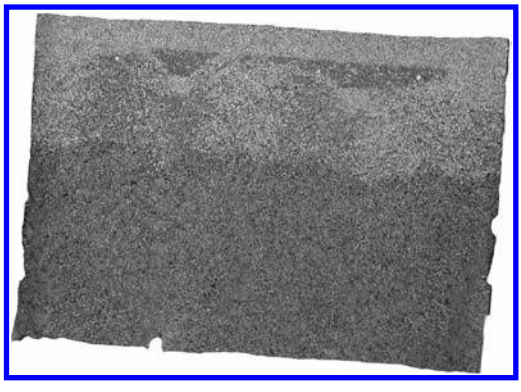

Figura 8. Foto ortogonal $Q = 23{,}4$ l/s.

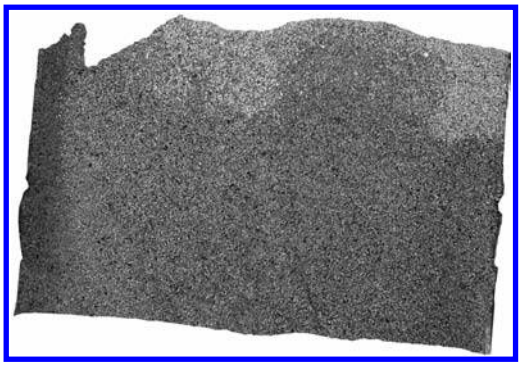

Figura 9. Foto ortogonal $Q = 26$ l/s.

5.1.3 *$D_{50} = 8{,}2$ mm talud $N = 2{,}5$*

El mecanismo de rotura para esta presa con talud $N = 2{,}5$ es un mecanismo con claro predominio del arrastre de partículas, las partículas son arrastradas de forma individual una vez que la tensión de arrastre vence la resistencia de la grava. El equilibrio de la partícula está gobernado por el balance de las fuerzas desestabilizadoras de sustentación y de arrastre, y de las fuerzas estabilizantes de gravedad e imbricación de las partículas.

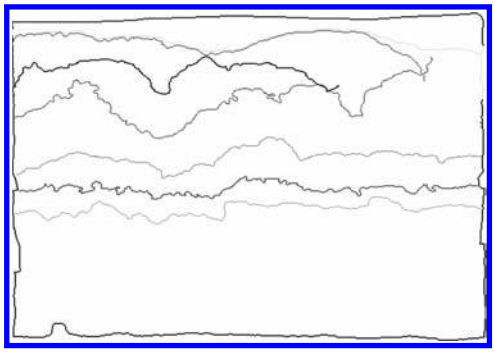

Figura 10. Avance en la rotura de la presa por escalones.

Cuadro 4. Datos del ensayo N°2.

$N = 1{,}75$ Escalón	Q (l/s)	h_a (mm)	h_b (mm)	A (m)	B (m)
E_1	5	164	8	0	0
E_2	8	224	8,8	0	0
E_3	10,7	268	11,6	0	0
E_4	12,6	293	12	0	0
E_5	14,7	319	12	0	0,287
E_6	17,4	345	12,8	0	0,608
E_7	20,7	379	16	0	0,913
E_8	23,4	400	16,4	0	1,039
E_9	26	415	16,8	0	1,181
Q_{ri}	23,6	–	–	–	1,05

Cuadro 5. Datos del ensayo N°3.

$N = 2{,}5$ Escalón	Q (l/s)	h_a (mm)	h_b (mm)	A (m)	B (m)
E_1	5,6	170	14	0	0
E_2	8	253	19	0	0
E_3	11	296	23	0	0
E_4	12,6	320	28	0	0,658
E_5	15	341	29	0	0,821
E_6	18	371	35	0,899	1,235
E_7	21	392	45	1,239	1,681
Q_{ri}	19,78	–	–	–	1,5

5.1.4 *$D_{50} = 8{,}2$ mm talud $N = 2{,}7$*

El mecanismo de rotura para esta presa con talud $N = 2{,}7$ al igual que la anterior es claramente un mecanismo de arrastre de partículas. A continuación se pone en una gráfica el camino de rotura de las 4 presas con diferente talud aguas abajo *N.*

En la gráfica se puede observar que una presa con un talud más tendido inicia la rotura con un caudal mayor.

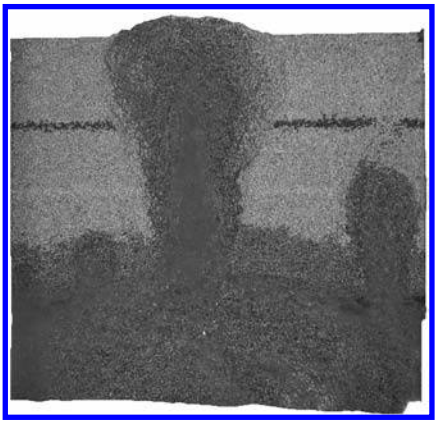

Figura 11. Foto ortogonal $Q = 21$ l/s.

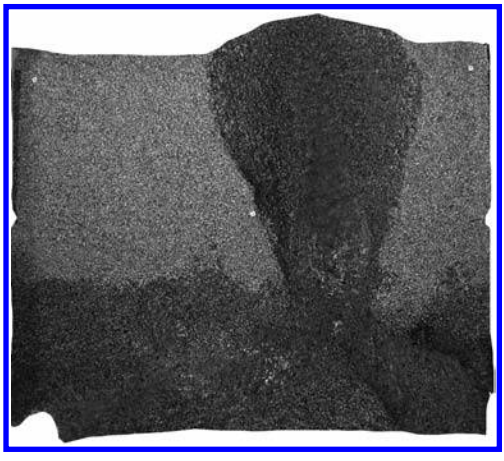

Figura 13. Foto ortogonal $Q = 23$ l/s.

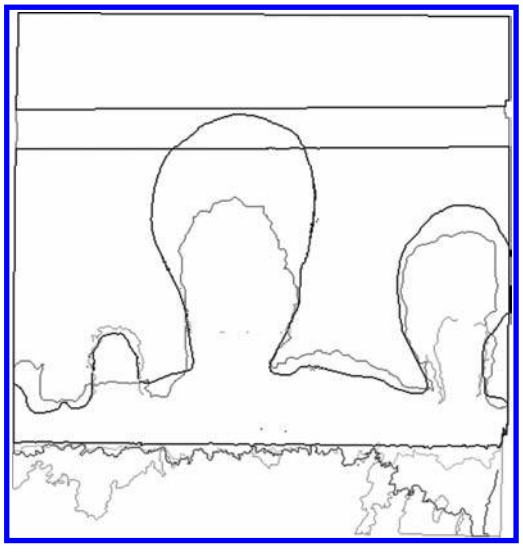

Figura 12. Avance en la rotura de la presa por escalones.

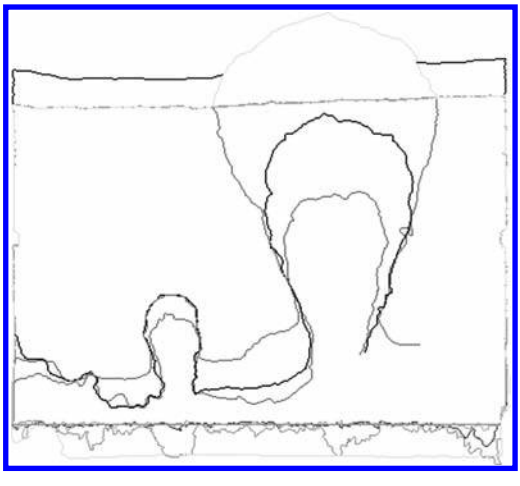

Figura 14. Avance en la rotura de la presa por escalones.

Cuadro 6. Datos del ensayo N°4.

$N = 2,7$ Escalón	Q (l/s)	h_a (mm)	h_b (mm)	A (m)	B (m)
E_1	5	193	15	0	0
E_2	7,6	252	21	0	0
E_3	11,3	313	25	0	0
E_4	14,3	348	30	0	0
E_5	17,5	380	30	0,936	1,142
E_6	20,2	359	30	1,148	1,539
E_7	23	–	–	1,314	2,053
Q_{ri}	20,64	–	–	–	1,62

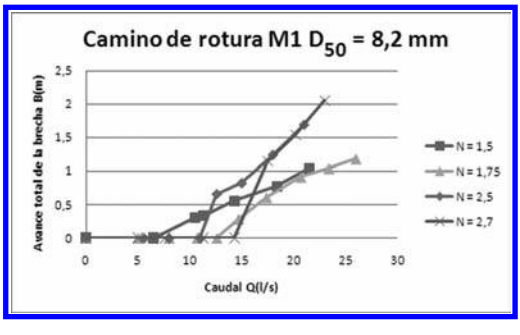

Figura 15. Camino de rotura M1.

5.2 *Material N°2 D_{50} = 12,64 mm*

Se realizaron pruebas para una presa de altura H = 1,0 m y dos taludes distintos aguas abajo, N = 2,2/3,0.

5.2.1 *D_{50} = 12,64 mm talud N = 2,2*

El mecanismo de rotura es el arrastre de partículas, se forma un canal de erosión que avanza hasta la coronación.

5.2.2 *D_{50} = 12,64 mm talud N = 3,0*

El mecanismo de rotura es el arrastre de partículas. Se forma un canal de erosión que avanza hasta la coronación.

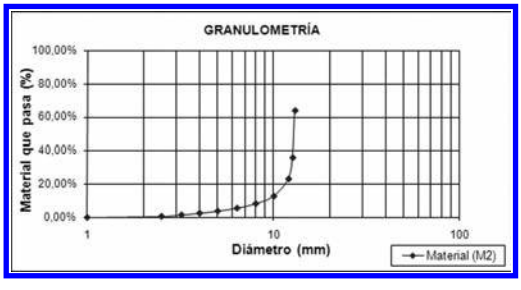

Figura 16. Material N°2.

Figura 17. Foto lateral Q = 38,47 l /s.

Cuadro 7. Datos del ensayo N°5.

N = 2,2 Escalón	Q (l/s)	h_a (mm)	h_b (mm)	A (m)	B (m)
E_1	10,35	300	17,2	0	0
E_2	19,04	431,8	24,4	0,3	0,3
E_3	30,63	550	29,9	1,06	1,19
E_4	38,47	607	31,9	1,51	2,21
Q_{ri}	38,39	784	–	–	2,2

5.3 *Material N°3 D_{50} = 35 mm*

Se realizaron pruebas para una presa de altura H = 1,0 m y cuatro taludes distintos aguas abajo, N = 1,5/2,2/2,5/3,0.

5.3.1 *D_{50} = 35 mm talud N = 1,5*

La rotura de esta presa con talud N = 1,5 está gobernada por el deslizamiento en masa del espaldón. Vuelve a observarse el proceso de deslizamientos sucesivos que progresa hasta llegar a la coronación.

5.3.2 *D_{50} = 35 mm talud N = 2,2*

El mecanismo de rotura para esta presa con talud N = 2,2 es un mecanismo combinado con

Figura 18. Foto lateral Q = 40,42 l/s.

Cuadro 8. Datos del ensayo N°6.

N = 3,0 Escalón	Q (l/s)	h_a (mm)	h_b (mm)	A (m)	B (m)
E_1	10,62	333	17,3	0	0
E_2	19,29	475	23,3	0	0
E_3	29,63	572	26,3	1,83	2,15
E_4	34,83	602	32,3	2,23	2,72
E_5	40,42	641	32,3	–	3
Q_{ri}	40,42	559	–	–	3

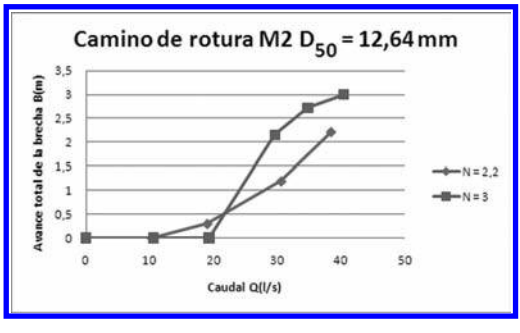

Figura 19. Camino de rotura.

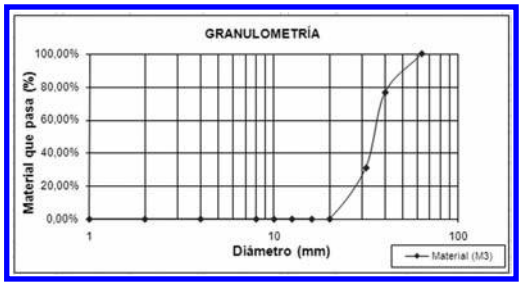

Figura 20. Material N°3.

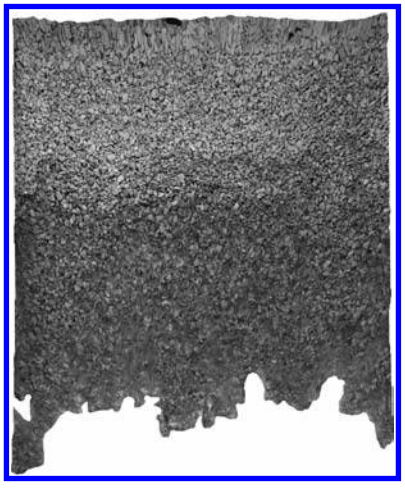

Figura 21. Foto ortogonal $Q = 90{,}68$ l/s.

Cuadro 9. Datos del ensayo N°7.

$N = 1{,}5$ Escalón	Q (l/s)	h_a (mm)	h_b (mm)	A (m)	B (m)
E_1	25,46	351	25,6	0	0
E_2	40,87	469,	33,4	0	0,615
E_3	51,75	537	39,4	0	0,71
E_4	69,07	632,	48,4	0	1,078
E_5	90,68	720	49,4	1,415	1,556
Q_{ri}	88,15	–	–	–	1,5

predominio del deslizamiento en masa. Se aprecian unos ligeros arcos en la zona deslizada, que denotan la presencia del mecanismos de arrastre.

5.3.3 $D_{50} = 35$ *mm talud* $N = 2{,}5$

El proceso de rotura de esta presa con talud $N = 2{,}5$ se observa un claro predominio de arrastre de partículas.

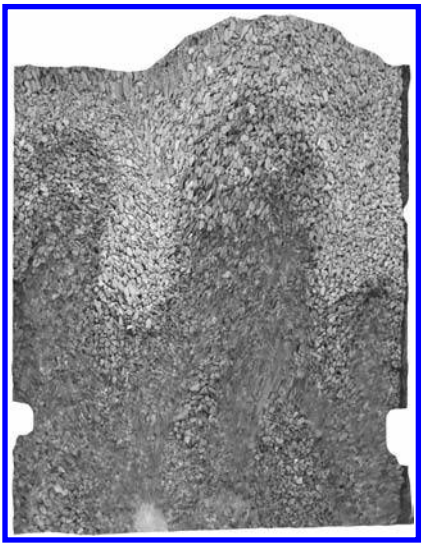

Figura 22. Foto ortogonal $Q = 90{,}68$ l/s.

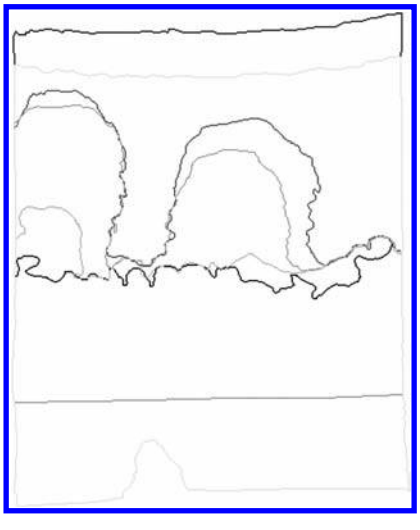

Figura 23. Avance en la rotura de la presa por escalones.

Cuadro 10. Datos del ensayo N°8.

$N = 2{,}2$ Escalón	Q (l/s)	h_a (mm)	h_b (mm)	A (m)	B (m)
E_1	27,16	411	20,6	0	0
E_2	38,99	509,5	26,4	0	0
E_3	50,42	555	29,4	0	0
E_4	69,07	686	37,4	1,177	1,319
E_5	90,68	767	48,4	1,625	2,066
Q_{ri}	94,56	–	–	–	2,2

Figura 24. Foto ortogonal $Q = 90{,}68$ l/s.

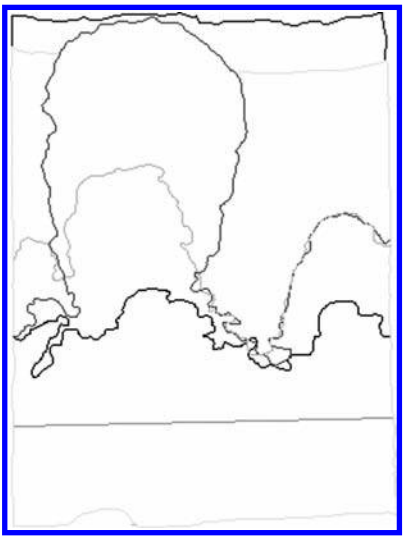

Figura 25. Avance en la rotura de la presa por escalones.

Cuadro 11. Datos del ensayo N°9.

$N = 2{,}5$ Escalón	Q (l/s)	h_a (mm)	h_b (mm)	A (m)	B (m)
E_1	26,80	432	25,4	0	–
E_2	35,80	498,2	30,2	0	–
E_3	49,10	598	35,4	0	0,935
E_4	81,69	669	40,4	2,09	2,438
E_5	90,68	746	40,4	2,422	2,721
Q_{ri}	83,66	899	–	–	2,5

5.3.4 *$D_{50} = 35$ mm talud $N = 3{,}0$*

El predominio del fenómeno es la erosión de partículas. El caudal de rotura es similar en los 4 casos. El inicio de la rotura se produce con un

Figura 26. Foto ortogonal $Q = 82{,}47$ l/s.

Figura 27. Avance en la rotura de la presa por escalones.

Cuadro 12. Datos del ensayo N°10.

$N = 3$ Escalón	Q (l/s)	h_a (mm)	h_b (mm)	A (m)	B (m)
E_1	31,87	475	29,2	0	0
E_2	48,23	602	38,2	0	0
E_3	60,57	684	39,2	0	1,378
E_4	75,54	737	44,2	2,139	2,25
E_5	82,47	736	44,2	2,39	2,842
Q_{ri}	84,32	693	–	–	3

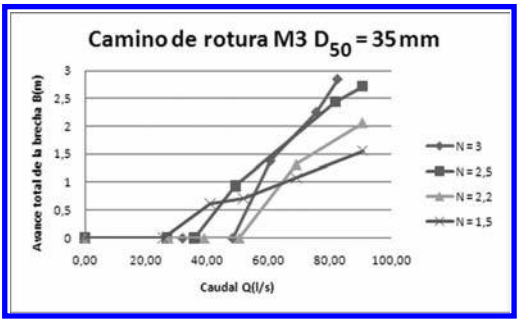

Figura 28. Camino de rotura M3.

menor caudal para una presa con el talud más inclinado.

6 PROCESO DE ROTURA

Los dos mecanismos de rotura que se aprecian en los ensayos son el deslizamiento en masa y el arrastre de partículas. Para los taludes intermedios se observa una combinación de ambos mecanismos.

6.1 *Arrastre de partículas*

En las presas de escollera, con espaldón de aguas abajo constituido por material granular no cohesivo, la resistencia al arrastre se produce principalmente por el peso de cada partícula o escollera considerada aisladamente y también por la imbricación de las partículas.

Dubuat en 1779 (Dubuat, L.G. 1779) introdujo por primera vez el concepto de esfuerzo cortante en el lecho y realizo experimentos sobre el movimiento de las partículas que lo componen. A partir de entonces se han propuesto varias expresiones, basadas en su mayoría en el criterio del esfuerzo cortante crítico, para predecir las condiciones del inicio de movimiento de partículas. La relación más conocida para describir el inicio de este movimiento fue propuesta por Shields en 1936, que determino el esfuerzo cortante crítico adimensional en función del número de Reynolds de las partículas. No obstante, el inicio del movimiento de partículas en las presas de escollera por sobrevertido tiene características diferentes: a) una gran pendiente y b)un flujo emergente del cuerpo de la presa.

Por ello, el movimiento de partículas en este caso particular ha de ser objeto de una investigación complementaria.

6.2 *Deslizamiento en masa*

La rotura por deslizamiento en masa se genera por la instalación de un campo de presiones intersticiales en el cuerpo de la presa como consecuencia del sobrevertido. El deslizamiento se inicia en el pie de presa y avanza aguas arriba hasta que la rotura llega a la coronación. A medida que avanza el deslizamiento en las fases previas, se amplía y asume una forma cuasicircular que mejora la estabilidad de la zona deslizada de la masa del espaldón. Entre los factores que influyen en el inicio del deslizamiento del espaldón están: el talud de la presa, el tipo y tamaño de la escollera y el grado de saturación del espaldón.

6.3 *Combinación de los mecanismos de rotura*

Para taludes intermedios se puede observar que existe una combinación de ambos mecanismos en el proceso de rotura. Toledo (Toledo 2004) ha estudiado la relación entre los dos mecanismos básicos de rotura, obteniendo conclusiones acerca de las circunstancias en las que la rotura se inicia por cada una de ellos. En 1961, Tinney y Hsu (Tinney, R.E. and Hsu, Y.H. 1961) establecieron pruebas de lavado en el laboratorio y concluyeron que el tipo de lavado es una función del tamaño de grano de los materiales, es decir, cuanto mayor sea el tamaño, más lenta es la velocidad de lavado. Simmler y Sametz (Simmler, H. and Sametz, L. 1982) estudiaron el desarrollo de la erosión de una presa por sobrevertido y propusieron una ecuación para la descarga a través de la brecha como una función del volumen de la brecha. Chinnarasri (Chinnarasri, C. 2000) investiga la erosión en diques homogéneos debido a sobrevertido mediante la realización de experimentos en laboratorio y mediante el uso de modelos numéricos (Tingsanchali, T. and Chinnarasri, C. 2001). Para el rango de la pendiente del dique aguas abajo entre 1V: 2.5H y 1V: 5H Concluyen que los parámetros esenciales que afectan el proceso de erosión son las propiedades del material de la presa, como el tamaño, la dureza, la erosionabilidad y la graduación del material que compone la presa. La geometría de la estructura, la ubicación del elemento impermeable, y la magnitud del volumen aguas arriba también afectan la tasa de erosión. Aunque este trabajo es interesante como punto de partida, su ámbito de aplicación es limitado y debe todavía esclarecerse el papel de algunos parámetros. Según los ensayos en modelo reducido realizados por el propio Chinnarasri, el ajuste del modelo matemático es todavía pobre, aunque sin duda mejor que el proporcionado por ejemplo por el programa comercial Dam Breach.

7 CAUDAL DE ROTURA

El caudal de rotura es un parámetro esencial, pues el elemento impermeable de la presa no resultará

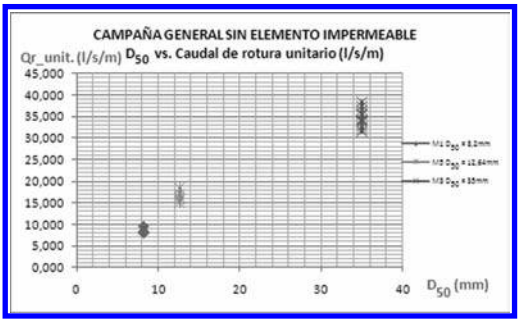

Figura 29. Diámetro medio Vs. Caudal de rotura unitario.

afectado por la rotura si el caudal de sobrevertido es menor que el de rotura y, en tal caso, puede excluirse la posibilidad de una rotura catastrófica.

Los resultados de los ensayos muestran claramente que el caudal de rotura depende del tamaño del material, con mayor valor en los materiales más gruesos, y de la altura de la presa, siendo mayor en las presas más altas. En cambio, no se observa una clara dependencia del talud de aguas abajo. Debe tenerse en cuenta que la precisión en la determinación del caudal de rotura es limitada. Por limitación de tiempo en una jornada, el número de escalones de caudal se limita a unos 6 ó 7, no disponiéndose de información para otros caudales que no correspondan a uno de los escalones elegidos. Consideramos que la precisión en la determinación del caudal de rotura ha sido insuficiente para captar la dependencia respecto del talud, que por otra parte es de segundo orden en comparación con la influencia del material y la altura de presa. Cabe pensar lo mismo en lo que se refiere a la dependencia del tipo de elemento de impermeabilización.

8 RESUMEN Y CONCLUSIONES

En todos los casos los movimientos de partículas aisladas o en grupo, tienen un carácter sensiblemente superficial. El inicio de la rotura se origina en el pie de presa y avanza hasta llegar a la coronación. En el caso de una rotura por deslizamiento en masa este crecimiento es homogéneo a lo largo de toda la presa. En el caso de fallo por arrastre, la rotura se concentra en uno o varios canales de erosión que progresan hasta llegar a la coronación. Un componente que parece determinante en la rotura es el nivel de saturación en el cuerpo de la presa. En todos los casos la rotura ha llegado hasta la coronación cuando la presión en la zona de coronación se encontraba entre el 50El talud aguas abajo de la presa determina el mecanismo de rotura y caudal de inicio de la rotura. La altura de la presa H y el tamaño medio D_{50} del material que compone la presa determina el caudal de rotura de la presa. De los resultados obtenidos se puede asegurar que el talud aguas abajo es determinante para el caudal de inicio de la rotura y para el nivel de saturación del espaldón a partir del cual se iniciará la rotura que avanzará hasta la coronación. El mecanismo de rotura a través del arrastre de partículas esta caracterizado por la aparición de uno o varios canales de erosión.

La profundidad de estos canales de erosión está relacionada con el tamaño medio del material, con mayor profundidad para un D_{50} menor. La ubicación del canal de erosión y el número de ellos en el caso de rotura por arrastre no ha tenido influencia en el caudal de rotura.

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Válvulas de asiento plano y paso circular: Criterios de diseño y análisis de su funcionamiento mediante modelación numérica

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ABSTRACT: One of the causes of loss of operation or disablement of the high-pressure outlets in dams throughout history has been the lack of strength in the valves that control them. This has been resolved partly through the use of sluice valves (Bureau type) or of hollow cone valves (Howell-Bunger). However the Bureau type valves have the disadvantage of needing transitions from circular to rectangular geometry and vice versa, often in short conduits. There are also other problems such as the loss of water tightness due to the accumulation of sediment in the bottom. In this regard INHISA has developed a new type of sluice valve that attempts to solve the aforementioned disadvantages.

Another difficulty for companies that manufacture these types of valves is the lack of tools that enable them to test the performance of new types before their practical use, especially as regards such critical aspects as vibration, air demand or the power necessary for their use in operational conditions.

The tests that are needed in order to check performance make developing new types or changing or adjusting them difficult. The effect of scale is very important in these cases and therefore it would be necessary to carry them out on actual valves under operational conditions.

Also, CIMNE (International Centre for Numerical Methods in Engineering), in collaboration with INHISA, is developing a numerical code based on the PFEM (Particle Finite Element Method) technique, within the framework of VADIVAP research, which is part of the National Applied Research Plan 2008–11. The main objective is to analyse the hydraulic and structural performance of the valve when faced with different demands. PFEM is an original numerical scheme that has successfully solved flow-structure interaction problems. It combines the Lagrangian approach with the solution of equations of finite elements using the generation of a net, which is updated as time passes. The code attempts, amongst other things, to incorporate the effect of the air-water mixture by simulating natural aeration and the demand of air generated downstream from the valve. The main characteristics of the new valve type, its primary practical applications as well as the first results of the numerical model using the PFEM method are shown in the communication.

1 INTRODUCCIÓN

Las conducciones hidráulicas están gobernadas porválvulas y compuertas de diversas clases, cada una con sus ventajas e inconvenientes según las aplicaciones para las cuales han sido diseñadas. De entre estos elementos destacan las válvulas de compuerta deslizante como las de tipo *Bureau*, nacidas para servir de elemento de seguridad y regulación en conducciones de agua a presión, tanto en desagües de fondo como en tomas de agua (CNEGP 1997).

Las válvulas *Bureau* son de solera plana y paso rectangular, habiéndose realizado importantes mejoras en su diseño en las últimas décadas; sin embargo, tienen el inconveniente de necesitar transiciones de longitud adecuada cuando la conducción que regulan es de sección circular. Por otro lado, las válvulas de compuerta deslizante de paso circular tradicionales presentan la desventaja de perder estanquidad con la acumulación de sedimentos en la recata inferior, lo que las hace no recomendables para ser usadas como elementos de regulación. En este sentido, INHISA ha mejorado el desarrollo y patentado una válvula de compuerta deslizante usando la tecnología de *U.S.B.R.* de las válvulas *Bureau* y la experiencia acumulada en el

diseño y fabricación de válvulas de paso circular, para desarrollar la denominada *Válvula de asiento plano y paso circular*^{p} (Marín and García 2002), que ha sido objeto de mejoras sustantivas en su diseño, fruto de años de observación y estudio de su comportamiento en aplicaciones reales.

Una vez alcanzado un alto grado de optimización mediante la observación de modelos reales, se hace necesario el estudio de las diversas características de diseño hidromecánico mediante herramientas más versátiles y económicas como son las numéricas. De esta forma, INHISA en alianza con CIMNE han desarrollado un modelo de una *Válvula de asiento plano y paso circular*^{p} mediante el *Particle Finite Element Method (PFEM)*, que es un innovador método numérico para resolver problemas de interacción fluido-estructura.

2 VÁLVULAS COMPUERTA DE ASIENTO PLANO Y PASO CIRCULAR^{p}

Las características de estas válvulas permiten su uso en desagües de presas como elementos de seguridad y regulación. Pueden ser accionadas en presiones desequilibradas, y presentan una estanquidad muy superior respecto a otras válvulas de cierres metálicos.

Al ser de paso circular, no requieren transiciones, haciéndola más adecuada en conducciones circulares. No presenta recata inferior, por lo que se evitan los problemas de acumulación de sedimentos, mejorando la estanquidad a lo largo del tiempo. Sus elementos principales son:

- *Obturador*: Elemento de cierre que, mediante su movimiento, consigue la apertura de la válvula.
- *Cuerpo de válvula*: Envolvente de la válvula en cuyo interior se aloja el obturador, tanto en posición de cerrada como de abierta. Dependiendo del tamaño de la válvula, el cuerpo se divide en cuerpo inferior y superior, que se unen mediante bridas.
- *Cilindro de accionamiento*: Apoyado en la cúpula, sujeta al obturador, permitiendo abrir y cerrar la válvula. Es accionado mediante un *grupo óleo-hidráulico* y controlado desde el *pupitre de mando*.
- *Indicador de posición*: Indica mediante un vástago anclado al obturador la posición de este.
- *By-pass*: Sistema alternativo de flujo de pequeño diámetro, que estando abierto permite el paso de agua de forma controlada desde aguas arriba hacia aguas abajo de la válvula.
- *Aducción de aire*: Sistema de alimentación de aire ubicado en el cuerpo inferior de la válvula aguas abajo del obturador, que permite disminuir las presiones negativas.

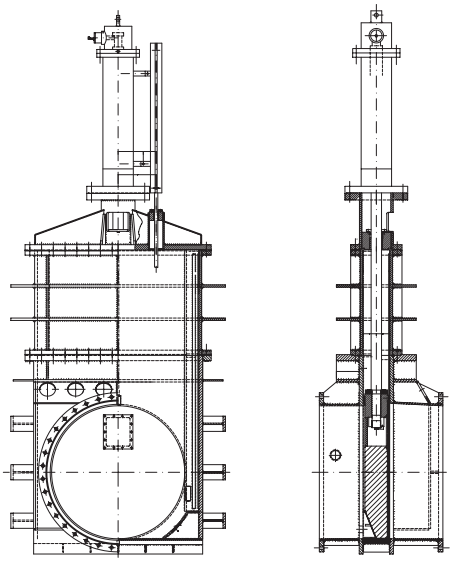

Figura 1. Válvula de asiento plano y paso circular^{p} de diámetro 1400 mm para una carga de 100 m.c.a.

Normalmente se emplean aceros no aleados para el cuerpo, cúpula y obturador, aceros aleados para las guías y vástagos, y bronces para los casquillos y cierres del obturador.

El diseño estructural se hace de acuerdo a la norma *DIN 19704: Hydraulic steel structures. Criteria for design and calculation*, bajo la hipótesis de que la válvula no esté embebida en el hormigón, de manera que sea auto-resistente.

Los primeros modelos de estas válvulas han demostrado, durante los años que llevan en funcionamiento, que son muy eficientes elementos de seguridad y regulación de las conducciones donde han sido implementadas, permitiendo su accionamiento con presiones desequilibradas, y eliminando los problemas generados por la acumulación de sedimentos en la solera.

Notables mejoras en los más recientes modelos han sido fruto de la observación y experimentación en modelos reales. Entre ellas destacan:

- *Cuñas de despegue de la vena*: Orientadas de tal manera que su espesor crece en el sentido de avance del flujo, y que está dispuesto en la pared interior del conducto y en la zona anterior de los orificios de las aducciones de aire según la dirección del flujo (Figura 2).
- *Limpiadores de recata*: Aprovechando el flujo de agua y el diferencial de presión generado entre la zona anterior y posterior del obturador, se orienta un chorro a las recatas del obturador para limpiarlo de los sedimentos que se pudieran acumular en los laterales (Figura 3).

- *Doble chaflán del obturador*: En la zona inferior del obturador hay un doble chaflán de 45 a 60 grados, que permite reducir la superficie de contacto entre el obturador y la solera (Figura 4).

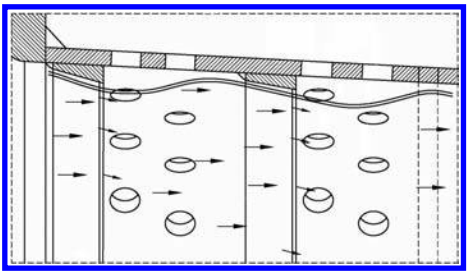

Figura 2. Detalle de los cuñas de despegue de lámina de la válvula.

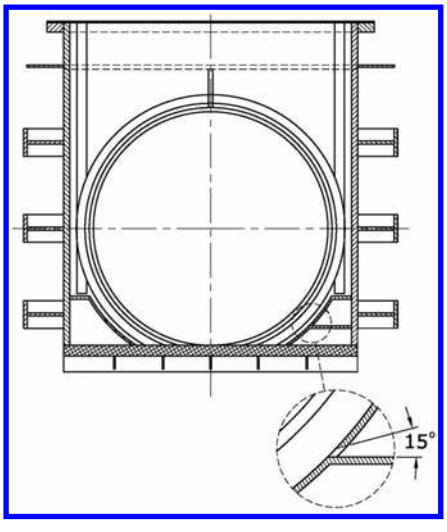

Figura 3. Detalle de los limpiadores de recata de la válvula.

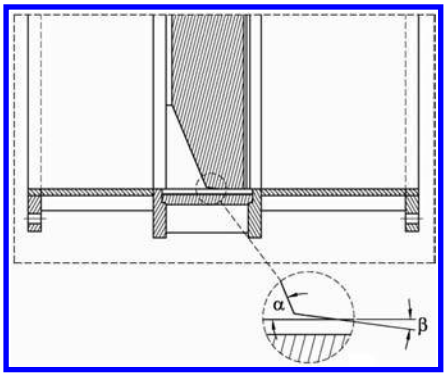

Figura 4. Detalle de la doble inclinación del obturador de la válvula.

3 CONSIDERACIONES DE DISEÑO

Además de la seguridad estructural, es de principal importancia en el diseño de válvulas la funcionalidad hidro-dinámica, que queda en gran medida determinada por características como la pérdida de carga, aireación del flujo o esfuerzos de elevación, todas estas de difícil evaluación. A falta de herramientas más fiables, se utilizan las tablas y recomendaciones de diseño para válvulas semejantes (USACE 2001), que dan aproximaciones a los valores reales de tan importantes parámetros.

3.1 *Pérdidas de carga*

Los caudales evacuados por los desagües están afectados por las pérdidas de carga en ellos. En estas conducciones cortas las pérdidas singulares en las válvulas son significativas, por lo que es necesario conocer sus coeficientes de pérdida de carga (Streeter 2001).

3.2 *Esfuerzos de elevación*

Estos esfuerzos estarán determinados principalmente por los esfuerzos de rozamiento, influyendo también el peso propio y los esfuerzos hidro-dinámicos como el down pull (Erbisti 2004), que pueden llegar a ser importantes en válvulas de grandes dimensiones y alta carga de agua, que requieren de un espesor de obturador significativo.

3.3 *Aireación*

Debido a su ubicación, las válvulas de desagües profundos soportan flujos a altas velocidades, lo que propicia la aparación de cavitaciones, vibraciones o posibles colapsos, con los consiguientes daños. Por tanto es necesario dotar a estos elementos de la adecuada aireación (Gutiérrez 1993), siendo este punto de especial importancia en esta investigación.

El fenómeno de la aireación en desagües de fondo no está suficientemente analizado, principalmente porque su estudio requiere realizar ensayos a escala real (Vischer and Hager 1999). En España se ha realizado un importante trabajo en este campo, aunque más centrado en la aireación en aliviaderos (Gutiérrez 1993). (Sharma 1976) hizo una clasificación de los tipos de movimiento aguas abajo de la compuerta, en el caso de no existir aireador en la solera:

1. Flujo pulverizado para aperturas inferiores al 10%, con un caudal de aire muy importante.
2. Flujo supracrítico en lámina libre, el típico de los desagües de fondo, con características del movimiento supracrítico, como es la aparición de ondas de choque y de una zona de mezcla agua-aire.

3. Flujo emulsionado, para sección casi llena con mezcla agua-aire.
4. Flujo con resalto en lámina libre, con importante arrastre de aire en el pie del resalto.
5. Flujo con resalto en carga, con transición a flujo en carga.
6. Flujo con resalto sumergido, debido a alta sumergencia, en el que no existe arrastre de aire.

Las Figuras 5 y 6 muestran los tipos de movimiento descritos ((Sharma 1976); tomado de (Gutiérrez 1993)).

En general, es conveniente que aguas abajo de la compuerta el agua circule en lámina libre, ya que de este modo se reduce de manera apreciable el riesgo de cavitación y disminuyen las vibraciones. Para ello, es necesario que exista un aporte suficiente de aire, que puede proceder de: a) los conductos de aireación situados inmediatamente aguas abajo de la compuerta y conectados con el exterior o: b) el punto de vertido de la conducción, desde donde se genera una corriente de aire en sentido inverso al del agua, que circula por la parte superior de la sección. La Figura 7 muestra un esquema general de la circulación del aire (Gutiérrez 1993).

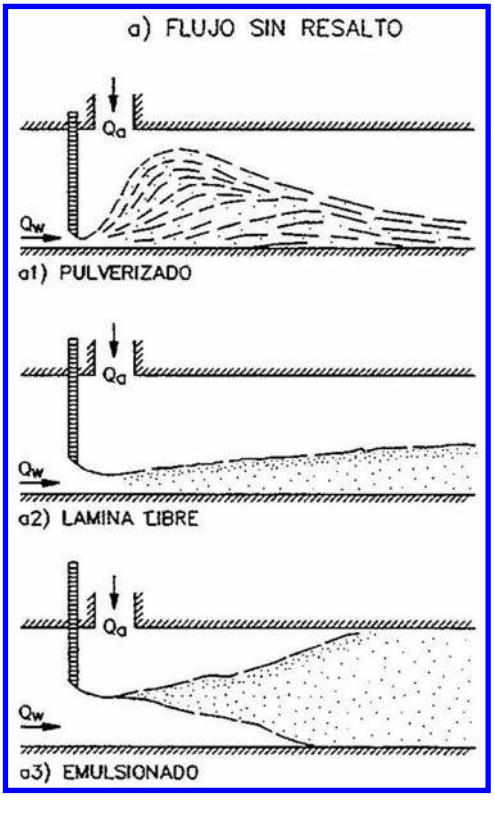

Figura 5. Tipos de movimiento en desages de fondo. Flujo sin resalto.

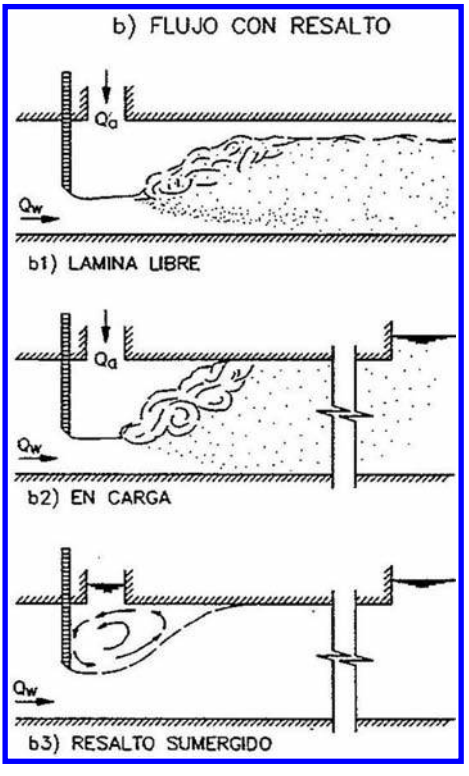

Figura 6. Tipos de movimiento en desages de fondo. Flujo con resalto.

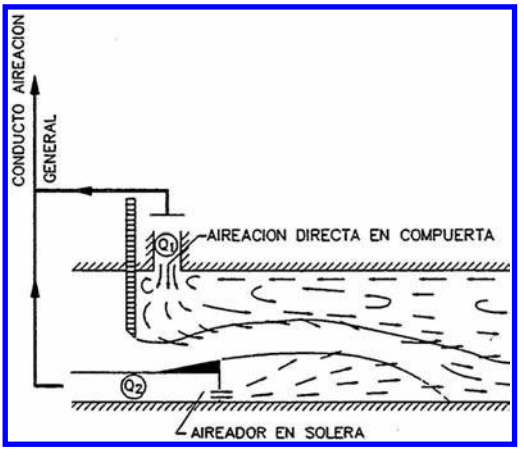

Figura 7. Esquema general de la aireación en desages de fondo.

4 METODO DE ELEMENTOS FINITOS Y PARTÍCULAS

En la técnica PFEM, tanto el dominio del fluido como del sólido se modelan utilizando una formulación Lagrangiana actualizada. Se utiliza

el Método de los Elementos Finitos (MEF) tradicional para resolver las ecuaciones de gobierno en ambos dominios, y para ello debe generarse una malla que los discretiza. Los nodos que discretizan los dominios del fluido y del sólido pueden interpretarse como partículas cuyo movimiento se sigue durante la solución transitoria.

La calidad de la solución numérica depende, obviamente, de la discretización utilizada, como sucede en el MEF estándar. Con PFEM es posible definir diferentes tamaños de malla en diversas zonas del dominio de análisis, para mejorar la solución en zonas donde ocurran grandes movimientos del fluido o de la estructura, o bien donde se quiera obtener mayor precisión.

Como se ha explicado, la formulación Lagrangiana permite seguir el movimiento de cada partícula individual del fluido (un nodo). Esto es útil para modelar la separación de las partículas del líquido del dominio principal del fluido y para seguir su movimiento como partículas individuales con una velocidad inicial y sometidas a las fuerzas de gravedad. En re-sumen, una solución típica con el PFEM sigue las eta-pas siguientes:

1. Discretizar los dominios del fluido y de la estructura con una malla de elementos finitos. En este trabajo se utiliza un esquema de generación de malla innovador basado en una teselación extendida de Delaunay (S.R. Idelsohn 2003), (S.R. Idelsohn 2004).
2. Identificar los contornos externos para los dominios del fluido y del sólido. En este trabajo se utiliza el método denominado Alpha Shape para la definición de los nodos del contorno.
3. Resolver las ecuaciones acopladas del movimiento Lagrangiano para los dominios del fluido y de la estructura. Calcular las variables de estado relevante en ambos dominios en cada paso de tiempo: las velocidades, la presión y las tensiones viscosas en el fluido, y los desplazamientos, las deformaciones y las tensiones en la estructura.
4. Mover los nodos de la malla a una nueva posición en función del tamaño del paso del tiempo. Esta etapa es típicamente una conse-cuencia del proceso de solución de la etapa 3.
5. Generar una nueva malla en caso necesario.
6. Volver a la etapa 2 y repetir el proceso de solución para el paso de tiempo siguiente.

5 DESARROLLO DEL MÉTODO

El método PFEM ya ha sido validado mediante comparación con resultados de ensayos de laboratorio realizados para el estudio de diferentes fenómenos hidráulicos, como el desagüe bajo compuerta, trampolines, o aliviaderos escalonados (A. Larese 2008).

Estos desarrollos previos de PFEM, junto con la experiencia acumulada en las aplicaciones ya realizadas en el campo de la ingeniería portuaria (proyectos SAYOM y DILAPE), son la base de partida del proyecto VADIVAP. El principal reto desde el punto de vista de la modelación numérica consiste en la consideración en el cálculo del la mezcla agua-aire. Se trata de un obstáculo importante desde el punto de vista de la programación, ya que supone considerar la interacción y mezcla de dos fluidos con densidades y viscosidades muy diferentes.

El trabajo desarrollado hasta el momento se ha organizado en dos líneas. Por una parte, se están haciendo pruebas en casos simplificados en dos dimensiones para estudiar la forma de considerar la mezcla agua-aire. Con PFEM se sigue el movimiento de los nodos durante la solución transitoria, lo que permite considerar la mezcla que se produce en el entorno de la superficie libre. Las Figuras 8, 9 y 10 muestran imágenes de uno de estos casos de prueba.

Obviamente, en este tipo de problemas la aproximación bidimensional no es suficiente, ya que debe considerarse el efecto de todo el contorno de la conducción, que en este tipo de válvulas es circular. Por ello, de forma paralela se están realizando pruebas con la versión anterior del código, sin la consideración de la presencia de aire, pero sobre el dominio completo tridimensional. Las Figuras 11 y 12 muestran instantáneas de uno de estos cálculos preliminares. Corresponde a un caso con apertura de compuerta de un 25%, y las dimensiones

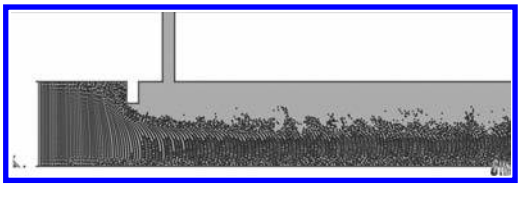

Figura 8. Imagen de un caso preliminar en dos dimensiones. Se observa cómo la zona de mezcla entre al agua (azul oscuro) y el aire (en azul claro) aumenta hacia aguas abajo.

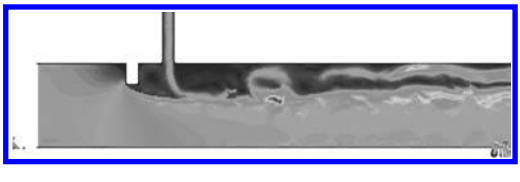

Figura 9. Otra imagen del mismo caso, en la que se muestra el módulo de la velocidad. Se observan los remolinos que se producen en la zona con aire, así como el caudal de aire que entra desde aguas abajo.

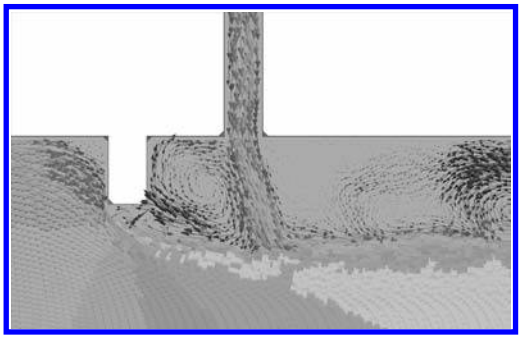

Figura 10. Detalle de la imagen anterior, donde se observa el caudal de aire que entra por el conducto de aireación.

Figura 11. Vista de uno de los casos 3D. Se muestra únicamente la capa de fluido. Los colores representan el módulo de la velocidad.

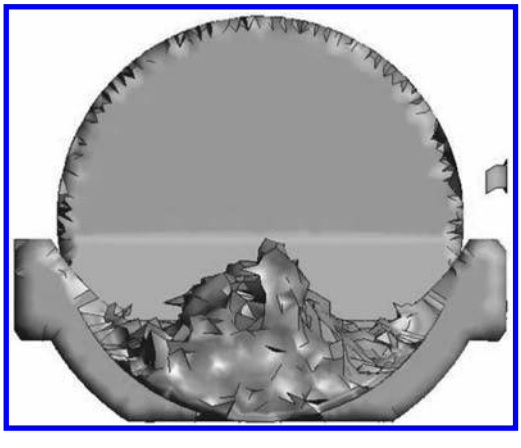

Figura 12. Vista frontal del caso de la figura anterior. Se observa la concentración de flujo en el centro del conducto.

son las reales de la válvula que se quiere estudiar (1400 mm de diámetro).

En estas primeras pruebas en tres dimensiones se aprecia que se reproduce el comportamiento observado en el funcionamiento normal, en lo que se refiere a la concentración del flujo en el centro del conducto para aperturas pequeñas.

6 DESARROLLOS FUTUROS

Una vez desarrollado el código para que pueda considerar el efecto del aire, se utilizará para los modelos 3D completos, en los que se introducirá además la geometría del sistema de aireación previsto para la válvula. De este modo se podrá comprobar su eficacia y optimizar su diseño, en caso de ser necesario.

Paralelamente, en CIMNE se está trabajando en la optimización del código de cálculo, de modo que sea posible afrontar problemas sobre dominios más grandes, con mallas más finas, y en un tiempo menor. De esta forma se aumentará en gran medida la utilidad de las herramientas numéricas.

El objetivo final de esta línea de investigación es la obtención de un código de cálculo de dinámica de fluidos, capaz de considerar el efecto de la aireación. Se trata de un problema que en la actualidad no está resuelto, y uno de los fenómenos que no es posible modelar mediante ensayos de laboratorio, a causa del efecto escala.

7 CONCLUSIONES

Se ha estudiado el efecto de las aducciones de aire en los conductos de desagües profundos. Se ha modelado y resuelto, en dos dimensiones, una válvula de asiento plano y paso circular^{p} bajo carga hidráulica usando PFEM, para obtener las primeras aproximaciones a resultados, y poder compararlos con los existentes en la literatura. Se ha modelado una válvula de asiento plano y paso circular^{p} en tres dimensiones calculando una aproximación de los resultados que no incluye el efecto de la aireación. Esta aproximación permite calibrar el software de cálculo. Una vez desarrollado el modelo numérico se pretende comparar los resultados con las mediciones realizadas en una válvula real trabajando en una presa.

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Problemática en el uso de un barranco como aliviadero de una presa

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RESUMEN: La presa de Huesna, situada en la sierra norte de Sevilla, se terminó de construir en 1990. El aliviadero se encuentra en la margen derecha, aprovechando la excavación de los materiales que se utilizaron para la ejecución del cuerpo de presa. El principal problema que se presenta es que el canal de descarga del aliviadero termina en un barranco natural con grandes inestabilidades en sus laderas, que se incrementan con la presencia del agua en régimen torrencial durante su funcionamiento. Los caudales naturales del barranco son muy inferiores al caudal de diseño del aliviadero. Recientemente se han realizado las obras de protección del barranco. Para estudiar el funcionamiento de estas obras se ha construido en el Laboratorio de Hidráulica del CEDEX un modelo físico reducido que reproduce las obras llevadas a cabo. Sobre el modelo se han ensayado las obras construidas y se han estudiado distintas alternativas para tratar de solucionar los problemas presentados. Los resultados obtenidos en el modelo, que posteriormente se han podido observar directamente en el vertido producido por el aliviadero este invierno, muestran la conveniencia de realizar este tipo de estudios para incorporar los resultados al diseño de las estructuras hidráulicas.

ABSTRACT: The Huesna dam, situated in the Northern mountain range of Seville, was completed in 1990. The spillway is located on the right bank, taking advantage of the excavation for material used for the construction of the dam body. The main problem confronted is that the spillway discharge channel ends in a natural precipice has considerable instabilities on the slopes, which increases with the presence of water in the torrential regime occurring during spillway operation. Ravine natural flow rates are much lower than the spillway design flow. Works for the protection of the ravine has recently been carried out. To study the functioning of these works reduced-scale physical model has been built in the hydraulics laboratory of CEDEX which reproduces the works carried out. The constructed works have built into the model and tested and various alternative designs to address the problems have also been tested. The results obtained in the model—which subsequently were observed directly in the discharge produced by the spillway this winter—confirm the advisability of performing these types of studies in order to incorporate the results into the design of hydraulic structures.

1 INTRODUCCIÓN

Es habitual efectuar ensayos de dispositivos hidráulicos como bombas, válvulas y difusores para verificar su comportamiento, o en conductos y canales para obtener datos a escala real. Sin embargo, existen numerosos casos en que el ensayo de la estructura real resulta imposible, algunas veces por razones técnicas y otras por razones económicas. En esos casos se recurre a la experimentación sobre modelo hidráulico.

Se denomina "prototipo" al ejemplar original y "modelo físico" a la analogía que reproduce en versión generalmente reducida del prototipo en estudio.

Es primordial diferenciar los términos "maqueta" y "modelo". En general, se trata de expresiones que se utilizan indistintamente de forma errónea.

Se puede decir que una maqueta es una reproducción a tamaño reducido de un monumento o construcción, o de una parte de éstos, realizado generalmente en cartón, madera u otro material.

Un modelo físico, además de reproducir a tamaño reducido el prototipo, pretende simular el comportamiento de una situación real. Sobre el modelo se pueden realizar mediciones y analizar las alternativas de diseño. Se suele añadir el adjetivo "físico" para diferenciarlos de los modelos matemáticos.

Para representar la realidad se debe cumplir no solo la semejanza geométrica, que se cumple en la construcción de maquetas por igualdad de ángulos y proporción de magnitudes lineales, sino también la semejanza cinemática (que tiene en cuenta tiempos y velocidades), y la semejanza dinámica exigiendo una única escala de fuerzas.

Esa "pequeña" condición suplementaria, de tan difícil cumplimiento en la mayor parte de los casos prácticos, marca la diferencia entre lo que es un "modelo" y lo que es una "maqueta".

Para que una maqueta se transforme en modelo, será necesario que la misma tenga "vida". Naturalmente, en el modelo sólo se ve la realización geométrica, pero esto solo es la punta del "iceberg".

2 ESCALAS Y LEYES DE SEMEJANZA

Siempre que se cumplan ciertas condiciones de semejanza, diferentes según los objetivos de cada estudio y las características de cada fenómeno interpretado, se podrán tomar mediciones de los parámetros de interés y analizar el comportamiento de variantes o alternativas de diseño que podrán ser aplicadas en el prototipo. Cuando esas normas se respetan, los procesos físicos ocurren en prototipo y modelo de manera cualitativamente análoga y cuantitativamente proporcional.

Los modelos físicos deben cumplir reglas muy estrictas para verificar las leyes de semejanza. Es necesario que exista una relación constante entre las fuerzas actuantes en prototipo y modelo, pero la fuerza de gravedad y el fluido, en nuestro caso el agua, siempre estarán presentes por igual en ambos. Por ello se recurre a la semejanza hidráulica que permite adoptar distinto criterio según la influencia en la semejanza de las distintas variables que intervienen en la ecuación general de la hidráulica.

Con el estudio del análisis dimensional y con la aplicación general del famoso teorema π de Buckingham en mecánica de fluidos se obtienen ocho monomios, cinco de ellos son los conocidos números de Euler, Froude, Reynolds, Weber y Cauchy.

La coincidencia de todos ellos en modelo y prototipo no es posible. Por ello, en los modelos hidráulicos se asegura la igualdad de uno de ellos según el tipo de fuerzas preponderantes en cada caso: gravitatorias F, viscosas R, tensión superficial W o elasticidad C, aparte naturalmente de las fuerzas de inercia. En la mayoría de los casos es necesario prestar especial atención a las fuerzas gravitatorias y viscosas.

En los fenómenos que tienen superficie libre es importante la influencia de las fuerzas de peso y puede despreciarse la influencia de la viscosidad si se trata; bien del estudio de vertederos o transiciones en régimen lento donde puede asimilarse a un fluido perfecto sin pérdida de energía. O bien si existe una apreciable pérdida de energía pero se pierde a través de una fuerte turbulencia, como en los resaltos, cauces fluviales o transiciones en régimen rápido. En estos casos la semejanza es la de Froude.

La teoría de modelos hidráulicos se basa en que las magnitudes longitud, masa y tiempo son independientes y fundamentales, con lo que se tendrían tres escalas, pero dado que la densidad del fluido es la misma en modelo y prototipo, dos escalas están relacionadas entre sí y solo existe la posibilidad de elegir una de ellas, normalmente la escala de longitud.

2.1 *Elección de la escala del modelo*

La escala del modelo se elige en función del espacio disponible, del caudal bombeado y del tamaño final del modelo que permita medir los parámetros deseados. La relación entre el resto de las escalas se realiza partiendo de la semejanza de Froude.

En el modelo es necesario además avalar la no influencia de las fuerzas viscosas garantizando un valor del número de Reynolds lo suficientemente elevado pasa asegurar el desarrollo de la turbulencia.

Una vez analizados los condicionantes del estudio, entre los que destacan la zona a representar y los caudales que deben ser suministrados al modelo, se eligió la escala geométrica 1/50 para el estudio en modelo físico reducido del aliviadero de la presa de Huesa. Esta escala resultó conveniente tanto por la construcción, como por la precisión de los registros, adaptación a la infraestructura del Laboratorio, facilidad de operación y duración de los ensayos.

3 PRESA DE HUESNA

Se trata de una presa de escollera con pantalla asfáltica de 73 m de altura situada sobre la Rivera de Huesna al norte de la provincia de Sevilla, entre los términos municipales de El Pedroso y Constantina. La presa se terminó en 1990 utilizando para su construcción los productos de la excavación de un collado artificial situado en la margen derecha del embalse donde finalmente se situó el aliviadero.

Con la apertura del collado se conectó, mediante un aliviadero de labio fijo, el embalse producido por la construcción de la presa sobre la Rivera de Huesna con el Barranco Carbonero.

Para la transportar los materiales procedentes de la excavación del collado hasta el cuerpo de presa se acondicionó el barranco, y una vez finalizada la construcción se dejó que el vertido por el aliviadero forjase el nuevo cauce.

Figura 1. Aliviadero y Presa de Huesna.

Figura 2. Desembocadura del Barranco Carbonero.

La primera avenida tuvo lugar en diciembre de 1996, el caudal punta no alcanzó los 100 m^3/s, pero fue suficiente para que arrastrase los productos de desecho de la excavación con los que se había allanado el barranco, iniciase el proceso de erosión de la roca situada en el lecho y la desestabilización de la ladera derecha del barranco.

El proceso de erosión no se estabilizó, sino que aumentó la inestabilidad de la ladera provocando además que el arrastre del material causara problemas en la desembocadura del barranco, unos 250 m aguas abajo de la presa, incluso taponando la salida de los desagües de fondo. En estas condiciones se decidió realizar unas obras de protección del barranco para detener el proceso de erosión.

En el Proyecto Base de Licitación se desarrolló un estudio de alternativas que planteaba dos posibles soluciones. La primera con circulación en régimen rápido a través de un canal revestido a lo largo de todo el barranco hasta la reincorporación al río. Y la segunda con actuaciones locales para tratar que la circulación se realizase en régimen lento. Finalmente se eligió la segunda de las alternativas.

4 PROTOTIPO

Como ya hemos dicho, se denomina "prototipo" al ejemplar original. En este caso el prototipo es la Presa de Huesna, más concretamente su aliviadero incluyendo todo el Barranco Carbonero hasta la su desembocadura, el cual ha sido el objeto del estudio en modelo físico.

4.1 *Descripción del aliviadero*

El aliviadero de construyó a la vez que la presa. Se trata de un perfil estricto de 2,5 m altura, 75 m de radio y 63 m de longitud de vertido, situado en la margen derecha del embalse. Su canal de descarga discurre por la trinchera abierta en el collado artificial del que se extrajo el material para la construcción de la presa. La trinchera excavada tiene una base de ancho variable de unos 50 m y pendiente del 1‰ con una longitud de 300 m.

4.2 *Obras en el Barranco Carbonero*

Al final de la trinchera, y coincidiendo con el inicio del barranco, se encuentra la denominada "obra de cabecera". Se trata de una estructura de hormigón escalonada de 4,25 m de altura que se adapta a la forma del cauce, pero con tan solo una longitud de vertido de unos 30 m frente al ancho de la trinchera que es superior a 50 m.

Para dirigir el flujo hacia la obra de cabecera se ha construido un pequeño muro de 1 m de altura. Este murete es rebasable para pequeños caudales haciendo que el vertido desde la trinchera al barranco no se produzca únicamente a través de la obra de cabecera, sino también sobre el terreno natural.

El Barranco Carbonero tiene una longitud aproximada de 900 m con una pendiente media del 7%. Para defender el barranco ante la erosión, se han construido cuatro azudes escalonados de 6 m de altura y cuenco de 10 m de longitud. También se han dispuesto diversos tramos de muros de gaviones para proteger las laderas, principalmente la ladera derecha, y se han intercalado cadenas para la estabilización del fondo del cauce.

Aguas abajo de la obra de cabecera el barranco tiene una pendiente superior a la media, entorno al 11%, coincidiendo con una pronunciada curva hacia la izquierda. Se trata, en principio, de la zona más problemática y en ella se han situado gaviones en la margen derecha y más de 15 cadenas en el cauce.

Desde el final de la curva la sección se va ampliando a la vez que va disminuyendo la pendiente hasta llegar al Azud n° 1.

Entre los Azudes 1 y 3 el trazado es más suave, tanto en planta como en alzado. La sección del cauce aumenta y la pendiente se reduce.

Figura 3. Curva aguas abajo de la obra de cabecera.

Figura 4. Zona entre los Azudes 1 y 3.

Entre los Azudes 3 y 4 se pueden diferenciar dos tramos, el primero similar al anterior, prácticamente recto y con sección amplia, pero en el segundo tiene lugar un estrechamiento brusco que coincide con una fuerte curva hacia la derecha que termina en el Azud 4. En esta zona no se han colocado gaviones, aunque si cadenas en el lecho del cauce.

Aguas abajo del Azud 4 tiene lugar la desembocadura del Barranco Carbonero en la Rivera de Huesna.

5 CAUDAL DE DISEÑO

El caudal de diseño inicial era de 350 m^3/s de acuerdo con los datos hidrológicos existentes hasta la fecha de redacción del Proyecto Base.

Posteriormente, en el Proyecto Variante, y conforme a nuevos datos, se adoptó como caudal de proyecto 550 m^3/s. Para fijar este caudal se consideró la avenida de 500 años de periodo de retorno considerando como hipótesis de partida que la cota inicial de la lámina de agua en el embalse era la establecida en las Normas de Explotación para un resguardo del 15%. Sin tener en cuenta esta limitación, el caudal evacuado sería de unos 700 m^3/s.

El problema que presenta este aumento del caudal es todavía mayor si consideramos que el desagüe se realiza a través de un barranco cuyos caudales naturales son muy inferiores a los de diseño del aliviadero.

6 OBJETIVO DEL MODELO FÍSICO

Durante las obras de protección del aliviadero de la Presa de Huesna se realizaron distintos cálculos hidráulicos del barranco utilizando un modelo matemático unidimensional. A los técnicos responsables no les pareció que con los resultados obtenidos quedase totalmente solucionado el problema y optaron por la realización de los ensayos en modelo físico reducido.

Así pues, el objetivo de la construcción y ensayo en modelo físico es observar el funcionamiento de las obras construidas para la protección del barranco donde descarga el aliviadero, y en su caso, el estudio de diferentes alternativas hasta alcanzar una solución razonablemente satisfactoria dentro de los condicionantes existentes.

7 CONSTRUCCIÓN DEL MODELO

En un principio estaba pensado construir y ensayar en modelo físico la obra de cabecera de protección del barranco y la zona aguas abajo de ésta correspondiente a la curva de pequeño radio y fuerte pendiente. Una vez vista y analizada la información se decidió, de acuerdo con los técnicos de la Agencia Andaluza del Agua, reproducir la totalidad de la obra entre el aliviadero y el último azud, pero finalmente se construyó una zona más amplia que incluye desde una parte del embalse próxima al aliviadero hasta un tramo de la Rivera de Huesna aguas abajo de la desembocadura del Barranco Carbonero.

Una vez decidida la zona a estudiar se determinaron los límites del modelo limitando las zonas de lecho móvil correspondiente al lecho presumiblemente erosionable. Posteriormente se realizó el proyecto constructivo del modelo y su adaptación a las infraestructuras del Laboratorio que permita la entrada y salida del caudal que debe ser suministrado, y por último se determinó el instrumental necesario para la toma de resultados.

Realizado el proyecto constructivo y fijado el emplazamiento en la nave de ensayos, se procedió al replanteo y a la construcción de los muros exteriores en fábrica de ladrillo. Posteriormente se realizó

el enfoscado y fratasado de muros y solera para terminar con la impermeabilización del recinto con doble capa de tela asfáltica. A continuación se ejecutaron las monteas o planos horizontales nivelados sobre las se colocaron las piezas correspondientes al aliviadero, obra de cabecera y azudes que previamente habían sido construidos en taller en mortero de cemento blanco.

Para reproducir el terreno considerado como lecho fijo, se imprimió en tiras de papel las curvas de nivel a escala de modelo, se dió forma a las varillas que las representan y finalmente se colocaron a la cota correspondiente dando lugar a un armazón topográfico. Este armazón se rellenó con arena, se compactó y se finalizó con un recubrimiento de mortero de cemento gris.

Sobre el propio modelo se construyeron los muros de gaviones en mortero de cemento blanco y se colocaron las cadenas realizadas en plástico en taller.

Las zonas consideradas como lecho móvil se realizaron formando una caja impermeable con profundidad suficiente para admitir las máximas erosiones previsibles que posteriormente se rellenó con el material elegido. En este modelo el espesor de la zona móvil se fijó en 10 cm. lo que equivale a 5 m de prototipo.

Simultáneamente a los trabajos de construcción del modelo se realizó el sistema de alimentación y retorno de caudal líquido. El control del caudal se realizó desde un aforador triangular en pared delgada.

Una vez finalizada la fase constructiva se situó en el modelo la instrumentación para la toma de niveles en numerosos puntos.

8 EXPERIMENTACIÓN

Normalmente el primero de los trabajos que se realiza sobre un modelo físico de estas características es, además de observar su comportamiento general, la obtención de la curva de capacidad del aliviadero. La curva que se obtuvo experimentalmente fue similar a las curvas teóricas facilitadas.

Posteriormente se iniciaron los trabajos de experimentación para una gama de caudales que varía entre los 50 y los 700 m^3/s sobre la solución inicialmente construida. Esta situación corresponde a las obras existentes en el barranco según muestra la Figura 5.

En un principio los ensayos se efectuaron utilizando un árido pequeño, de 1 cm. de diámetro, en la zona delimitada como lecho móvil, pero a la vista de los resultados obtenidos, donde las erosiones superaban las máximas previstas, se optó por repetir los mismos ensayos utilizando un árido más grueso, entre 2 y 3 cm. de diámetro.

En ambos casos se observó que, tanto la obra de cabecera como los azudes no producían la perdida de energía deseada, y que la circulación del flujo no se realizaba en régimen lento como estaba previsto, por lo que las erosiones que se producían eran superiores a las esperadas.

El murete situado en la margen izquierda de la obra de cabecera era rebasado para caudales entorno a los 75 m^3/s, y para este mismo caudal los cuencos de los respectivos azudes eran insuficientes.

Para el caudal de diseño, 550 m^3/s, incluso con el árido grueso, se observaron movimientos en el lecho aguas abajo de la obra de cabecera y el flujo circulaba por encima de la cota superior de gaviones en varios tramos. La lámina de agua alcanzaba la coronación de los cajeros de los azudes.

En la zona donde el cauce alcanza su máxima amplitud, en las proximidades del Azud 3, se forma una barra con el material procedente de las erosiones producidas en los tramos situados aguas arriba.

En el último tramo, en la curva situada entre los Azudes 3 y 4, el flujo circula preferentemente por la margen izquierda exterior a la curva, debido a la velocidad que alcanza en esa zona, existiendo un desequilibrio importante entre los niveles alcanzados en ambas márgenes. También en este tramo se observó el movimiento del lecho móvil del cauce.

Figura 5. Modelo terminado.

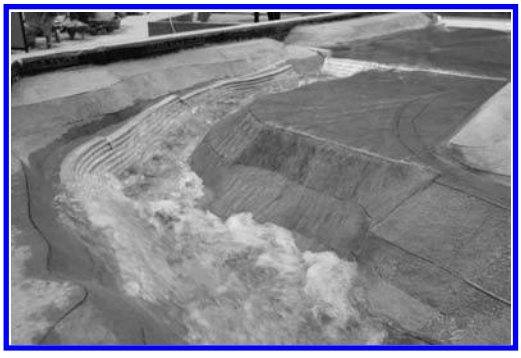

Figura 6. Ensayo para la avenida de diseño.

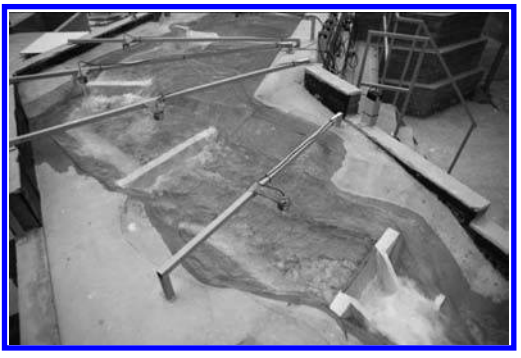

Figura 7. Alternativa en régimen lento.

9 ALTERNATIVAS ESTUDIADAS

A la vista de los resultados obtenidos durante la experimentación de la solución inicial, se estudiaron distintas posibles alternativas. Las primeras enfocadas a conseguir una circulación en régimen lento, tratando de disminuir la velocidad del flujo y por lo tanto las erosiones.

En el modelo se construyeron más azudes intermedios, se aumentó la cota de los existentes y se ampliaron sus cuencos, pero ninguna de estas alternativas resultó satisfactoria. Los niveles que alcanzaba la lámina de agua eran elevados y podrían afectar a la estabilidad de las laderas.

Descartadas las alternativas en régimen lento se inició el estudio de las alternativas con la circulación del flujo en régimen rápido. Fueron muchas las variantes estudiadas para un canal ya que los condicionantes eran importantes.

Se pretendía que la nueva obra incluyese en la medida de lo posible las existentes, obra de cabecera y cuatro azudes, ya que su demolición podía afectar aún más la estabilidad de las laderas.

También se deseaba que la rasante del canal se ajustase lo más posible al terreno. Debido a la escasez de material los rellenos deberían ser los menores posibles, pero sobre todo las excavaciones para no afectar a las laderas que se encuentran en situación más frágil.

Por último el canal no debería menoscabar los muros de gaviones que están al pie de las laderas, principalmente la ladera derecha.

En este contexto no resultaba fácil encajar una solución. En un principio se diseñó un canal de sección rectangular de 12 m de anchura para que coincidiese con el ancho de salida de los azudes. Esto no era posible en toda la longitud del barranco, ya que en la primera curva este ancho afectaría a los gaviones, y en la última las excavaciones afectarían a la ladera izquierda. Para el paso del canal por los azudes se eliminó el primer escalón de cada uno de ellos y se rellenaron el resto de los escalones.

Figura 8. Deslizamiento junto al Azud n° 2.

Figura 9. Gaviones y deslizamiento.

10 ALTERNATIVA PROPUESTA

Sobre el primer diseño se realizaron numerosas modificaciones para tratar de eliminar las ondas que se producían en el canal y para definir la altura de los cajeros.

Las zonas que más complicaron el diseño fueron la embocadura aguas arriba de la obra de cabecera, la primera y la última curva, así como el cuenco para reincorporación del flujo en régimen lento al cauce principal. En el caso de las dos curvas mencionadas, el canal se diseñó con una sección menor al no ser aconsejable aumentar las excavaciones.

Finalmente se diseñó un canal con seis secciones tipo diferentes, la última de ellas en falso túnel.

En el inicio del canal es necesario crear una transición desde la trinchera, con sección trapecial de 50 m de base, hasta la primera sección tipo, de sección rectangular de 10,32 m. Esta primera sección coincide además con la curva de pequeño radio y fuerte pendiente por lo que el flujo tiende a circular por el exterior de la curva. Para intentar minimizar este efecto se colocó una pila central a lo largo de toda la sección.

Tabla 1. Secciones tipo del canal.

Sección tipo	Ancho (m)	Altura cajeros (m)
1	10,32	8
2	variable	5
3	12	Dcho. 5–izq. 6
4	12	5
5	variable	6
6	9	6

Figura 10. Alternativa propuesta.

Figura 11. Cuenco de restitución al cauce principal.

Desde la salida de la primera curva la sección va aumentado su anchura hasta alcanzar los 12 m. La altura de los muros varía según el tramo e incluso dentro de una misma sección como ocurre en la sección tipo 3. Esto es debido a que coincide con una curva y el cajero exterior tiene una altura 1 m por encima del cajero interior. Aguas abajo de la curva de la sección tipo 3 destaca la colocación de una pila en el centro de la sección de 15 m de longitud. Esta pila disminuye la formación de ondas en los tramos situados de aguas abajo.

La sección tipo 5 vuelve a tener un ancho variable para llegar con un ancho de 9 m a la última curva, donde después de numerosos tanteos se optó por una sección rectangular en falso túnel.

A la salida del túnel se ha situado un cuenco de remanso para la restitución del flujo al cauce principal. Los cajeros son divergentes para adaptarse a la geometría de la desembocadura del barranco y se han dispuesto dos filas de dientes que permiten disminuir su longitud.

11 AVENIDAS EN PROTOTIPO

En el invierno de 2010 tuvo lugar una nueva avenida. Según los técnicos responsables de la presa, el caudal máximo vertido por el aliviadero fue pequeño, aproximadamente de 30 m^3/s. A pesar de ello se han producido daños en el primer tramo de gaviones aguas abajo de la obra de cabecera, coincidiendo con la curva de mayor pendiente.

Los arrastres del material del lecho y la afección a las cadenas de hormigón se asemejan a lo previsto en los ensayos en modelo reducido.

Al decir que los resultados se asemejan en modelo y prototipo es necesario puntualizar algunos detalles:

Primero, los ensayos se han realizado para caudales entre 50 y 700 m^3/s, muy superiores al caudal vertido por el aliviadero durante la avenida.

Figura 12. Avenida registrada el pasado invierno de 2010.

Figura 13. Efectos de la avenida en el prototipo.

Segundo, en el modelo se ha representado el lecho móvil con dos tamaños distintos de árido, pero en ningún caso es representativo de la granulometría del lecho ni de su cohesión.

Tercero, se ha supuesto un espesor de 10 cm. de lecho móvil, equivalente a 5 m de prototipo, a lo largo de todo el modelo, sin considerar la profundidad a la que se encuentra la roca en cada punto.

Cuarto, tanto los gaviones como las cadenas están representados en el modelo como elementos fijos no deformables.

Con estas consideraciones se puede decir que las erosiones que se apreciaron en el modelo durante los distintos ensayos son válidas tanto en su posición como en su forma, pero no lo son en cuanto a su magnitud.

12 CONCLUSIONES

Los modelos físicos reducidos son una técnica que nos ayuda a resolver problemas de ingeniería hidráulica que no son abordables por otros métodos. Tanto los cálculos teóricos, como la modelación matemática pueden ofrecer resultados erróneos debido a la dificultad de los contornos y a la complejidad de los fenómenos de turbulencia.

En este modelo en concreto donde la geometría es compleja, con variaciones importantes de sección y de pendiente que dan lugar a cambios de régimen, el ensayo en modelo físico facilitó el estudio del problema.

Los modelos físicos, aunque representan un alto coste económico y necesitan tiempo para su construcción y ensayo, pueden suponer un ahorro importante en el momento de realizar la obra.

Es aconsejable que los estudios en modelo físico se realicen en fase de proyecto ya que permiten introducir modificaciones o mejoras antes del inicio de la obra.

Normalmente es difícil poder confirmar el buen funcionamiento de los modelos con datos registrados en prototipo ya que se ensayan situaciones extremas que rara vez tienen lugar. Pero en los casos en los que estos escenarios se producen es de gran valor poder contar con esta información.

Queremos agradecer a los técnicos de la Agencia Andaluza del Agua su colaboración durante todas las fases del estudio, especialmente por la información proporcionada sobre la última avenida registrada, ya que el contraste de los resultados obtenidos en el modelo físico con las observaciones realizadas en prototipo nos ha permitido confirmar algunos de los detalles estudiados.

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Reservoir siltation

Dam Maintenance and Rehabilitation II – Romeo García et al. (eds)
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Restore reservoir capacity in a sustainable and very cost effective way

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ABSTRACT: Almost every reservoir is affected by sedimentation. WCD estimated that almost 1% of worldwide storage capacity is lost due to this effect. Even the actual new build of reservoirs does not level out overall storage decrease. Dredging and disposing reservoir sediment is extremely expensive. On the other hand the sediment which is missing downstream of reservoirs leads to erosion damages, substrate deficits and ground water problems. An innovative technical approach makes reservoirs permeable for sediment on a very cost effective basis. Incoming and already settled sediment is transferred downstream involving new equipment operation techniques. This restores overall sediment transport to a near nature state with overall benefits to ecology and the rivers morphological state, giving the operator back the desired reservoirs operational range. The new technical sediment management approach is transferable on almost any range of plants, small to large and run-of-river to pump storage. It is supported by environmental organizations and authorities and scientifically guided by University of Technology Aachen.

1 MORPHOLOGY

1.1 *The untapped river*

Like water and organisms also solid components are essential ingredients of every river. This solid fraction mostly is of natural origin and finds its way via erosion processes into the water body. Starting at the rivers source solid components usually begin their career as eroded massive rock. Transport processes changes rock size first to gravel size bedload and furthermore to fine-grained sediment. This sediment settles at the river floor. Depending on the local current this sediment does also come loose again by erosion effects. In middle and lower parts of natural rivers sedimentation and erosion usually balance each other (Figure 1).

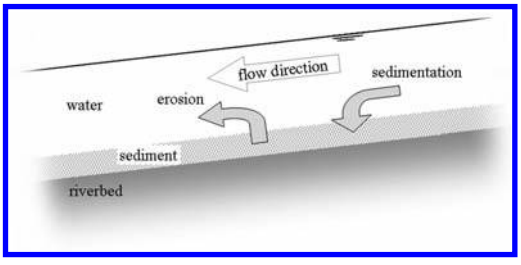

Figure 1. Sedimentation/Erosion equilibrium in natural rivers.

1.2 *Hydraulic changes*

A natural river changes it morphology fundamentally when installing a dam or other barrage across a river section. Permeability is still given for water and mostly for aquatic life by discharge elements and fish ladders. The extended cross-section upstream of a barrage leads to low current velocities and therefore to profoundly more sedimentation while erosion is minimized. Therefore a barrage often is a massive barrier for sediment transport (GWSP 2000). Many reservoirs are affected by massive sedimentation and consequently a loss of storage volume (Figure 2). The same is true for pump storage reservoirs.

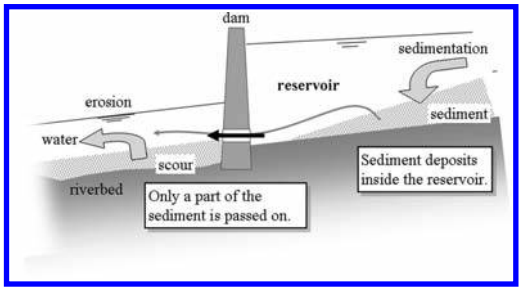

Figure 2. Reservoirs change the sediment balance profoundly.

1.3 *Operational consequences*

Operational restrictions caused by sedimentation do not come overnight. They increase gradually over long time periods, usually years. Therefore many operators get used to live with these restrictions and consider them as normal, though they are not. Even worse, they grow over time if no counteraction is taken. The World Commission of Dams estimates that annually 0.5 to 1% of global storage volume is lost by sedimentation. Without further action one quarter of all dams in the next 25 to 50 years lose their storage function by sedimentation (WCD 2000). Given this scale it is surprising what little attention is often given to the field of sediment monitoring, control and operational solutions.

Besides the effects working within a reservoir the sedimentation also affects the downstream river. To compensate for the 'missing' solid fraction in the aqueous phase the river erodes its bed. For that reason souring is widely common below river barrages. Souring can reach a level where structural failure of hydraulic structures or bank collapse happens. Restoration projects are accordingly costly.

2 SOLUTION ATTEMPTS

In many cases of sediment issues action is taken not until problems are in far progressed stage. Previously solutions attempts are briefly outlined below.

2.1 *Former procedures*

2.1.1 *Opening the base outlet*

Sedimentation processes usually start at the upstream entrance into the reservoir which typically is the most distant point from the dam axis. As sedimentation increases the settled sediments eventually reach the dam. To prevent the dams discharge elements from plugging the operator is now forced to flush the base outlets periodically (e.g. every six months). If neglected the equipment will become inoperable within short term because the sediment covers the gear. The tremendous runoff generated by opening the outlets erodes the sediment right upstream of the intake. The eroded sediment is transported downstream in short time and at a high rate (Figure 3).

Performing this method is quite simple and requires no further physical facilities. However, with this technique a tremendous quantity of water for power generation or water supply is lost. Furthermore, significant amounts of sediment are moved into the downstream river section in a very short time period. This can lead to negative morphological and ecological effects.

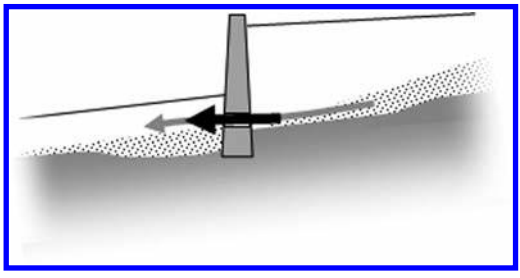

Figure 3. Opening the base outlet.

Anyway, the method is applicable only when sedimentation already reached the dam line. Opening the base outlet then only leads to a dissipation of sediments in close vicinity of the outlets. The overall operative range of the reservoir is not restored.

2.1.2 *Manual dredging*

Another procedure is the manual removal of sediments. Here the sediment is excavated by suction dredgers, hydraulic excavators or—after lowering the reservoirs water level and initial draining—wheel loaders (Figure 4). After the often expensive removal and transport the sediment has to be stored on separate drainage fields for years or decades in order to reduce the water content. Thereafter it may be used as covering material for simple ground work applications. The relatively high percentage of organic ingredients (usually 2 … 30%) prevents a use as a ground construction material even after dredging. In many cases, landfill is required.

The procedure allows for a thorough cleaning of reservoirs, but at exorbitant costs. Expenses consist of the sediment dredging activities, a plant/reservoir shutdown of several months as well as transport and dump expenditures which are in a million dollar range even at small reservoirs.

2.1.3 *Sludge dredging and disposal downstream*

This method involves several suction dredging campaigns where sediment is dug, transferred over the barrage and dumped downstream (Figure 5). For good reason this is not allowed in most countries. The loss of great amounts of flushing water for power production or irrigation may be even acceptable. The transfer of large sediment quantities into the downstream river section however causes a massive intrusion into the river morphology and ecology.

2.2 *Impact on river morphology*

When opening the base outlet the downstream river is often blocked for months. In any case the

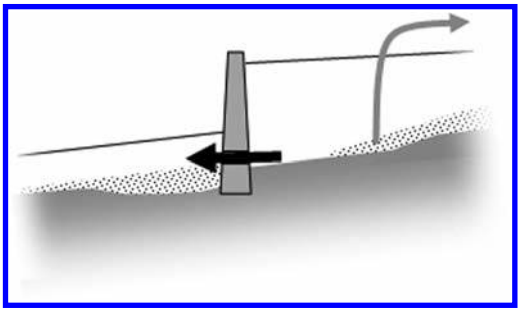

Figure 4. Manual dredging.

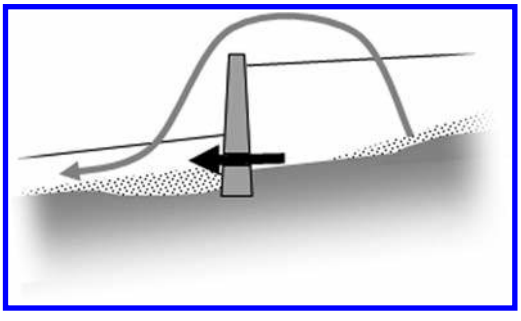

Figure 5. Sludge dredging and disposal downstream.

morphology is changed significantly. With manual removal, the sediment is extracted permanently from the water body. This leads to increased erosion in the lower reaches and therefore also affects the river, albeit with opposite effect. To compensate for only the most severe effects of erosion, e.g. more than 1 Mio. tons of artificial sediment are added to the river Rhine each year.

Dredging and disposal downstream usually is not compatible with river morphology due to the large amounts that need to be transferred in a short period of time. So all previously described existing methods are problematic in a morphological point of view.

2.3 *Ecological aspects*

Via morphological effects also ecological topics are directly concerned in all described dredging methods. In addition the emptying of the reservoir required for a manual dredging leads to a destruction of the current reservoir's fish population. With sludge dredging and direct disposal downstream the underwater benthos structure is widely destroyed within a few days. Opening the base outlet or even failing to do any maintenance on the reservoir and risking siltation ends in a profoundly long-term change of the developed ecosystems.

2.4 *Legal constraints*

The above presented methods usually require permission by local or regional authorities before starting any physical activity. For permission the authorizing body will take into account appropriate guidelines as European Water Framework Directive or U.S. sediment acts. Usually the facility owner has to expect more or less extensive obligations for project execution even if permission is granted.

When digging the sediment and extracting it from the water body the contractor or plant operator usually becomes owner of the removed material. If critical ingredients are detected, the extracted sediments have to be dumped as contaminated material at enormous expense. This also applies in the case that the critical components found are of endogenous and thus natural origin (e.g. certain heavy metals).

2.5 *Economic evaluation*

The opening of the base outlet initially appears as a cheap solution. However, the above-mentioned constraints have to be taken into account. If opening the base outlet seems to be a suitable solution, the degree of sedimentation already is far advanced with often severe operational restrictions on reservoir management. The base problem and also the operational restrictions are not resolved by this method as are resulting financial implications. Both will continue because sediment is eroded only in the upstream area near to the barrages outflow, not at the far end of the reservoir where most sediment is found. In addition it has to be taken care that no damage compensation liabilities are caused when discharging large quantities of sediment downstream in a short period of time.

The latter aspect also is valid for suction dredging with direct disposal into the downstream river section. For this reason, this variant in most cases is not licensable or generally unlawful.

Excavation of sediment from the reservoir usually is exorbitant expensive due to high direct construction costs, enormous disposal expenditures and long plant/reservoir downtime. For this reason, operators choose this option usually only when threatened by plant/reservoir loss.

3 ALTERNATIVES

As a promising alternative to the previous discussed methods a new approach is paving its way. According to Water Framework Directives or Sediment Acts a barrage should not only be penetrable for water and fish, but also for sediment. The general

approach is to bring the balance of sedimentation and erosion in a river back to a naturally acceptable or aspired degree.

3.1 *Required hardware*

The process is set up by devices allowing continuous and controlled transfer of sediment within the reservoir in a relatively small scale but permanent mode. Key element is an automatically working vessel with a suction dredging system installed that can be diesel or preferably electric driven (Figure 6). To allow for an exact positioning the vessel is directed by tractor cables. Sediments are loosened by a suction head, pumped towards the reservoir's outlet and dumped in front of the outflow elements. The vessel gradually strikes the reservoir floor until the complete surplus sediment is removed.

The newly dumped sediments are eroded by the hydraulic discharge and therefore carried out of the reservoir, passing turbines or outlet valves. Both types of outflow elements are able to handle water flow with some degree of sediment load. The sediment transfer rate can be adapted to the outflow rate and parameters to guarantee a compatible process speed. For an environmentally sound project implementation the time span should be set to months or years, depending on local conditions. A commercially competitive project performance is still assured by a high degree of automation.

The process is also applicable in pump storage reservoirs. When starting up a new pump storage plant, the newly created upper basin does not contain any sediment. After several years of operation the owner will frequently find a substantial degree of sedimentation, often limiting the plants operational parameters and commercial benefit. By using the process described here the initial state can be restored in an environmental friendly and commercially effective way.

As mentioned above the sediment leaves the reservoir via the usual outflow organs consisting of turbines, spillways, weirs etc. Wear on the discharge elements (e.g. turbine blades) keep on a normal range unless you increase sediment content to a considerably higher level. Anyway, process benefit should easily outrange slightly higher rehab costs or useful turbine blade coating.

Special care should be given to the turbines shaft main sealing. In many hydro power facilities a seal flushing with clear water already is installed. In these cases usually nothing else has to be done to prevent the seal from additional wear. At older turbines it is easy to add this feature—or the operator simply accepts a slightly higher wear on the sealing.

3.2 *Impact on river morphology*

When attaching a dam to a natural waterway the river morphology changes considerably. By inventing the described method the natural morphology is widely restored. During the time span required for the reservoirs surplus sediment removal the concentration of solid components in the outflow will be slightly higher than long time average. Even this is a natural condition as sediment processes often are not continuous. Natural sediment layers of several meters are absolutely common and may be washed away in a single flood.

The scale of the 'removal time span' depends on sediment characteristics such as grain size/specific surface, organic constituents, salinity, cation exchange capacity, pH-Value and temperature and has to be adapted to local constraints (Schweim 2005). Other aspects will be investigated in cooperation with the Institute for Hydraulic Engineering and Water Resources Management of Aachen University of Technology.

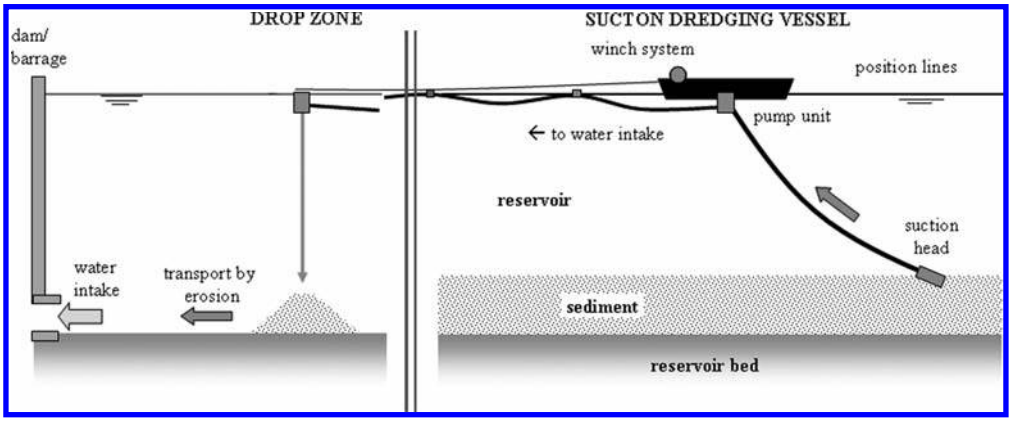

Figure 6. Continuous sludge dredging and transfer downstream.

3.3 *Ecological benefit*

Before starting the permanent sediment transfer the relevant ecological aspects have to be determined for the reservoir and the downstream river. The process has a positive impact on changes in storage volume. The periodically striking of the reservoir floor with the suction dredging tools however affects the benthos structure. This is similar to natural erosion. As the suction dredging never affects the overall floor at once and the process is stretched on a wide time span the reservoirs ecosystem should compensate the interference even better than natural erosion events.

At the beginning of a sediment transfer the downstream river will experience a higher than average but not unnatural sediment concentration. When transitional effects fade out the river will be set back to a natural sediment equilibrium. This is the aspired ecological state where natural conditions are restored and many regulatory frameworks fulfilled.

The ecological benefit ranges far downstream, restoring the desired nutrition properties of sediment and also re-establish the flux of terrestrial sediment to the global coastal ocean. Without this flux, sweet water reservoirs are threatened by infiltration of ocean salt water, as Global Water System Project published (Figure 7).

3.4 *Legal aspects*

In many countries the method requires no permission of local authorities because it represents a way of maintaining the hydraulic system, the active sediment transfer takes place only inside the reservoir, a natural condition is restored and the appropriate sediment acts/water directives are fulfilled. Due to these benefits the process has actively been supported by local authorities so far.

In fact, the effects provided by the described process are asked for in many ecological programs. The 'European Water Framework Directive' identified Sediment deficit as a major problem and in its Annex 5 actively promotes sediment permeability for rivers in there entire length including reservoirs.

In any way the operator should consult the relevant administrative bodies in advance. The same procedure is significant for patent legislation (PCT 2008).

3.5 *Economic advantages*

Since sediment is simply transferred back to the natural flow using an intelligent mechanism, no disposal or dump costs occur. Even the necessary technical facilities are relatively small and cost effective. The process benefit is achieved by continuous small and sustainable changes on a large timescale instead of large interventions in a short period. The cost of the necessary components is a fraction of the cost of conventional methods. Plant shutdown is limited to a few hours for installation instead of months for excavation works. The former storage volume and operating range is restored in some months or few years—permanently.

For these reasons, in many applications the method is economically very attractive, even if

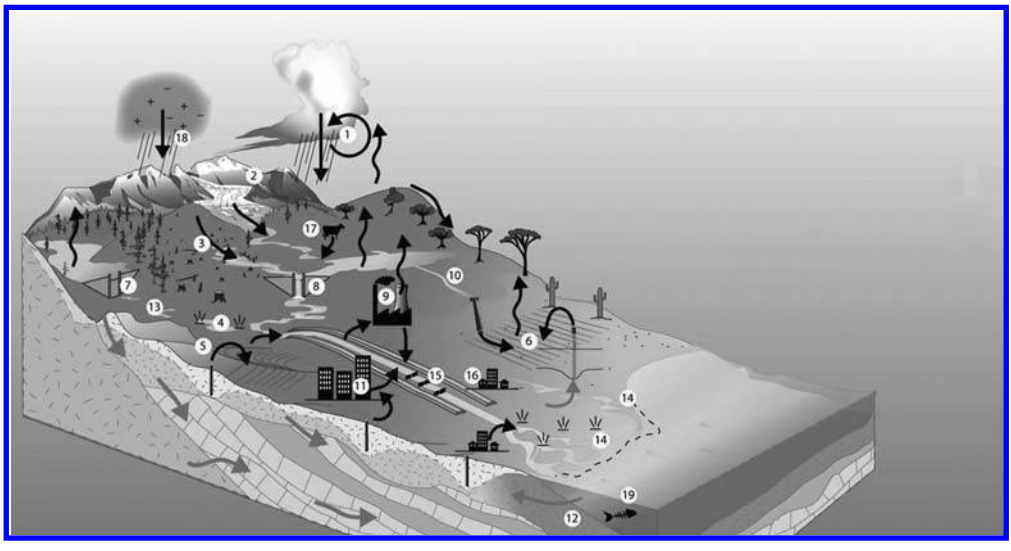

Figure 7. Sediment importance for ecosystems (GSWP 2009).

no immediate action in terms of solving sediment problems is obligatory. Additional benefit can be gained if the above described improvement of the ecological status is financially honored as in Germanys Renewable Energies Act (EEG).

For reservoirs that are almost inoperable due to sedimentation the operator should first think of restoring the existing facility by sediment transfer instead of an additional new reservoir, the latter including major construction work and flooding of additional land.

4 OPERATIONAL RESTRICTIONS

The process can be applied on almost any reservoir size, e.g. from relatively small German Ehreshoven II reservoir with 200 × 400 m in size (Figure 8) to multi-km² lakes. Of course equipment needs to be adjusted for different dimensions. For small applications fully automated vessels are available which completely fit into a 20'-container. Large applications require sectional built vessels with manned operation.

Figure 8. Aerial view on Ehreshoven Reservoir with visible sedimentation previous to sediment transfer.

5 CONCLUSIONS

With the presented method a transfer of sediment is performed in a very elegant way. Ecological permeability of dams is improved and sediment problems in reservoirs are not only eliminated, but permanently resolved. The procedure is in line with and supported by U.S. Sediment Acts, European Water Framework Directive and similar guidelines. Moreover, the process is very cost effective, so that a fast spreading can be expected—a rare and ideal combination of economical and ecological benefit. A rising number of applications will give the chance to analyze additional aspects which will be especially interesting for river sciences.

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Estudio en modelo numérico del transporte sólido en suspensión del río Segre en la cola del embalse de Ribarroja de Ebro

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RESUMEN: Se presenta el módulo de transporte de sedimentos en suspensión implementado en el modelo bidimensional Iber (CEDEX, CIMNE, GEAMA de la UdC y Grupo de Investigación Flumen UPC) y una aplicación práctica del mismo en el río Segre. El modelo Iber permite realizar el estudio del transporte de sedimentos en suspensión a lo largo del tiempo; a partir de las condiciones hidrodinámicas (calados y campo de velocidades en dos dimensiones) se obtienen las zonas de erosión/deposición, así como la concentración de sedimento del agua. Después de realizar la calibración del módulo se modelizó la dinámica sedimentaria para el tramo del Río Segre comprendido entre la confluencia con el río Ebro y un kilómetro aguas arriba de la población de Mequinenza, dónde se pretende construir un espigón para proteger la zona del campo de regatas de la sedimentación.

ABSTRACT: A suspended sediment transport module implemented in the two-dimensional Iber model (CEDEX, CIMNE, GEAMA of the UDC and Research Group Flumen UPC), and a practical application of it on Segre river are presented. Iber model allows the study of the transport of suspended sediment over time, from hydrodynamic conditions (depths and velocity fields in two dimensions). Results showing areas of erosion/deposition and sediment concentration can be obtained. After performing the calibration of the model, sediment dynamics are modeled along a reach of Segre River from the confluence with Ebro River and a kilometer upstream of the town of Mequinenza, where a breakwater is planned to protect a rowing race channel.

1 INTRODUCCIÓN

La confluencia de los ríos Ebro y Segre se produce justo aguas abajo de la presa de Mequinenza, ya dentro de la cola del embalse de Ribarroja de Ebro. Por encargo de Ingeniería 75 y propuesto por la Confederación Hidrográfica del Ebro, el Grupo Flumen de la Universitat Politècnica de Catalunya realizó el estudio de la hidródinámica y la dinámica sedimentaria de la cola del embalse de Ribarroja para el estudio de la estabilidad del campo de regatas de la población de Mequinenza.

El estudio se realizó con el módulo de transporte de sedimento del modelo numérico Iber (www.iberaula.es), desarrollado por el CEDEX (Centro de Estudios y Experimentación en Obras Públicas), el CIMNE (Centro Internacional de Métodos Numéricos en Ingeniería), el GEAMA (Grupo da Enxiñeria da Agua e do Medio Ambiente) de la Universidade da Coruña y el Grupo de Investigación Flumen de la UPC (Bladé et al., 2009).

1.1 *Motivación del estudio*

La intención de este estudio, por encargo de Ingeniería 75 y propuesto la Confederación Hidrográfica del Ebro, era estudiar la dinámica sedimentaria para el tramo de río Segre comprendido entre la confluencia con el río Ebro y un kilómetro aguas arriba de la población de Mequinenza, dónde se pretende construir un espigón para proteger el campo de regatas de la sedimentación.

Debido a las características del sedimento en la zona del estudio (López et al., 2010; Grupo de Investigación Flumen, 2009) se consideró oportuno estudiar el transporte de sedimento en suspensión y no únicamente el transporte por arrastre de fondo.

Por este motivo se desarrolló un nuevo módulo de transporte de sedimentos en suspensión implementado en el modelo Iber.

Seguidamente se utilizaron datos reales de batimetrías del tercio superior del embalse de Ribarroja de Ebro realizadas en 2007 y 2008 (Arbat et al., 2010; Grupo de investigación Flumen, 2009) para calibrar el módulo de transporte de sedimentos y poderlo utilizar para el estudio de la evolución hidrodinámica del tramo final del Segre (en el entorno de la población de Mequinenza).

2 EL MODELO IBER

Iber (desarrollado conjuntamente por el CEDEX, el CIMNE, el GEAMA de la UdC y el Grupo de Investigación Flumen UPC) es un modelo numérico de simulación de flujo turbulento en lámina libre en régimen no-permanente, y de procesos medioambientales en hidráulica fluvial.

El rango de aplicación de Iber abarca la hidrodinámica fluvial, la simulación de rotura de presas, la evaluación de zonas inundables, el cálculo de transporte de sedimentos y el flujo de marea en estuarios.

El modelo Iber consta de 3 módulos de cálculo principales: un módulo hidrodinámico, un módulo de turbulencia y un módulo de transporte de sedimentos. Todos los módulos trabajan sobre una malla no estructurada de volúmenes finitos formada por elementos triangulares o cuadriláteros. En el módulo hidrodinámico (constituyente de la base de Iber), se resuelven las ecuaciones de aguas someras bidimensionales promediadas en profundidad (ecuaciones de Saint Venant 2D). El módulo de turbulencia permite incluir las tensiones turbulentas en el cálculo hidrodinámico, pudiéndose utilizar para ello diferentes modelos de turbulencia para aguas someras con diferente grado de complejidad. En la versión actual se incluyen un modelo parabólico, un modelo de longitud de mezcla y un modelo k-ε.

El módulo de transporte de sedimentos resuelve las ecuaciones de transporte de fondo y transporte turbulento en suspensión, calculando a partir del balance de masa de sedimento la evolución de la cota de fondo. Estas ecuaciones son empíricas, obtenidas para casos particulares, por lo que los resultados obtenidos con ellas serán sólo válidos para el análisis de tendencias.

2.1 *Módulo de transporte de sedimento en suspensión 2D*

El módulo de transporte de sedimentos en suspensión utiliza los campos de velocidades, calados y de turbulencia proporcionados por los módulos hidrodinámico y de turbulencia (Bladé et al., 2009). El transporte de sedimentos en suspensión se modela mediante una ecuación promediada en profundidad, la ecuación implementada en el código es la siguiente:

$$\frac{\partial hC}{\partial t}+\frac{\partial hU_xC}{\partial x}+\frac{\partial hU_{xy}C}{\partial y} = \frac{\partial}{\partial x_j}\left(\left(\Gamma+\frac{v_t}{S_{c,t}}\right)h\frac{\partial C}{\partial x_j}\right)+\frac{\partial D_{sx}}{\partial x} + \frac{\partial D_{sy}}{\partial x}+(E-D) \quad (1)$$

Donde C es la concentración de sólidos en suspensión promediada en profundidad, U_x, U_y son las dos componentes de la velocidad horizontal promediadas en profundidad, v_t es la viscosidad turbulenta, Γ es el coeficiente de difusión molecular de sólidos en suspensión, y $S_{c,t}$ el número de Schmidt (relaciona el coeficiente de difusión turbulenta de momento con el coeficiente de difusión turbulenta de sólidos en suspensión).

Los términos D_{sx}, D_{sy} modelan la dispersión de sedimento en suspensión debido a la no homogeneidad del perfil de velocidades y de concentración de sedimento en la dirección vertical. Normalmente su efecto se desprecia en los modelos 2D de aguas someras.

Los términos E y D modelan respectivamente la puesta en suspensión de sólidos que se encuentran en el fondo (resuspensión de sedimento) y la deposición de sólidos en suspensión en el fondo del lecho. Su diferencia representa un balance, y por lo tanto un acoplamiento, entre carga de fondo y carga en suspensión. El cálculo de la evolución morfológica se realiza, resolviendo la ecuación de conservación del sedimento (ecuación de Exner) que considera la variación de la cota de fondo debido al gradiente del caudal sólido y también la Erosión (E) y deposición (D). La primera depende de la concentración de equilibrio:

$$E = W_s C_a^* \quad (2)$$

Donde W_s es la velocidad de sedimentación de las partículas sólidas y C_a* la concentración de sólidos instantánea. Mientras que la deposición depende de la velocidad de sedimentación de las partículas y la concentración de materia en suspensión:

$$D = \alpha W_s C \quad (3)$$

Dónde α es un el cociente entre la concentración cerca del fondo y la concentración media en la profundidad).

2.1.1 *Concentración de sedimento en suspensión en condiciones de equilibrio*

Iber implementa 3 formulaciones para el cálculo de la concentración de sedimento en suspensión en condiciones de equilibrio: Van Rijn (1987), Smith (1977), Ariathurai y Arulanandan (1978).

La tercera es la que se utiliza para el presente estudio por ser la más indicada para suelos cohesivos; en ella se hace depender la erosión de la diferencia entre la tensión tangencial (τ_b) y una tensión tangencial crítica de inicio de erosión (τ_c), así como de un valor M_2 representativo de la tasa de erosión (que sería la tasa de erosión cuando $\tau_b = 2\tau_c$):

$$E = M_2\left(\frac{\tau_b - \tau_c}{\tau_c}\right) \tag{4}$$

2.1.2 *Velocidad de sedimentación de las partículas*

La velocidad de sedimentación de las partículas se calcula en función de su diámetro (D_{50}) como (van Rijn, 1987):

$$W_s = \frac{R \cdot g \cdot D_{50}^2}{18 \cdot \upsilon} \quad si \quad D_{50} < 10^{-4} m \tag{5}$$

$$W_s \frac{10 \cdot \upsilon}{D_{50}}\left(\sqrt{1 + 0.01 \cdot D_*^3} - 1\right) si \quad 10^{-4} m < D_{50} < 10^{-3} m \tag{6}$$

$$W_s = 1.1 \cdot \sqrt{R \cdot g \cdot D_{50}} \quad si \quad 10^{-3} m < D_{50} \tag{7}$$

Donde D_* es el diámetro instantáneo.

2.2 *Puesta a punto del módulo de transporte de sedimentos*

Por encargo de la Confederación Hidrográfica del Ebro y con la finalidad de estudiar la dinámica sedimentaria del embalse de Ribarroja (Arbat et al., 2010; Grupo de investigación Flumen, 2009), entre finales de octubre y principios de noviembre de 2007 se realizó una campaña batimétrica de precisión mediante sonda multihaz y a partir de ella se obtuvo el modelo digital del fondo.

Para analizar la dinámica sedimentaria del tercio superior del embalse de Ribarroja (donde tiene lugar la entrada del río Segre), en diciembre de 2008 se realizó una nueva batimetría de los primeros 10 kilómetros de embalse. Mediante la comparación de las dos batimetrías se pudo cuantificar el sedimento acumulado/erosionado a lo largo del periodo, así como analizar su distribución espacial (Arbat, 2010).

En la Figura 1 se puede observar que en los 4000 m situados inmediatamente aguas abajo de la entrada del Segre se produjo un volumen de erosión sensiblemente igual al que se sedimentó en los 3000 m siguientes.

Un volumen de unos 170.000 m^3 de sedimento fue desplazado unos 4 km aguas abajo. En el periodo comprendido entre ambas batimetrías el único episodio de avenida importante se presentó en el Ebro entre finales de mayo y principios de junio de 2008, con un caudal medio diario máximo de 1600 m^3/s, siendo superior a los 500 m^3/s durante unos 12 días.

Como que el evento de avenida fue únicamente en el Ebro, el Segre no aportó sedimento que pudiera reponer el que se había movilizado aguas abajo de la confluencia.

El módulo de transporte de sedimento en suspensión se calibró de manera que introduciendo el modelo digital de la batimetría de 2007, los parámetros del sedimento de la zona de la confluencia (López et al., 2010) y el hidrograma de la avenida de mayo-junio de 2009 los resultados después de la avenida fueran similares a los obtenidos con la batimetría realizada en 2008 (Arbat et al., 2010). Se calibró tanto con la magnitud de las profundidades de erosión/deposición (del órden de pocas decenas de centímetros) como con la distribución espacial (Figura 2) del sedimento. Los valores estimados de los parámetros fueron $M_2 = 0.000005$ m/s y $\tau_c = 1$ N/m^2.

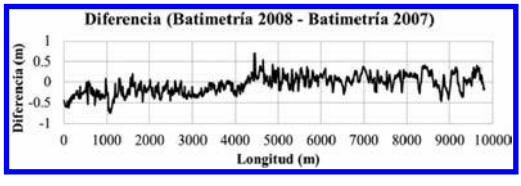

Figura 1. En este gráfico se puede observar la diferencia de los dos modelos digitales del fondo de embalse (batimetría 2008—batimetría 2007). Se puede ver que para los primeros 4000 m los valores son negativos, por lo que indican erosión; a partir de aquí, la diferencia es positiva, indicando zonas deposicionales. Diferencia en metros.

Figura 2. Zonas de erosión (blanco) y zonas de deposición (negro). Se puede observar claramente que los primeros 4000 metros aguas abajo de la confluencia Segre-Ebro son mayormente erosivos, mientras que aguas abajo la tendencia es deposicional.

3 APLICACIÓN AL ESTUDIO DE LA HIDRODINÁMICA DEL SEGRE EN EL ENTORNO DE MEQUINENZA

Después de realizar la calibración se modelizó la hidrodinámica y la dinámica erosiva/deposicional para el tramo del río Segre comprendido entre la confluencia con el río Ebro y un kilómetro aguas arriba de la población de Mequinenza (la zona de estudio se puede observar en la Figura 3), dónde se pretende construir un espigón para proteger de la sedimentación la zona del campo de regatas. En el estudio se simularon distintas geometrías y longitudes de espigón para varios caudales de avenida y distintos niveles de embalse, posteriormente se compararon con la situación actual.

3.1 *Modelo digital del Segre en el entorno de Mequinenza*

Los trabajos de medida del fondo del embalse se realizaron en agosto de 2009, siguiendo la misma metodología que en las campañas batimétricas 2007 y 2008 (Arbat et al., 2010; Grupo de investigación Flumen, 2009). Para el tramo de estudio se utilizaron dos ecosondas (multi y monohaz de doble frecuencia) instaladas en una barca.

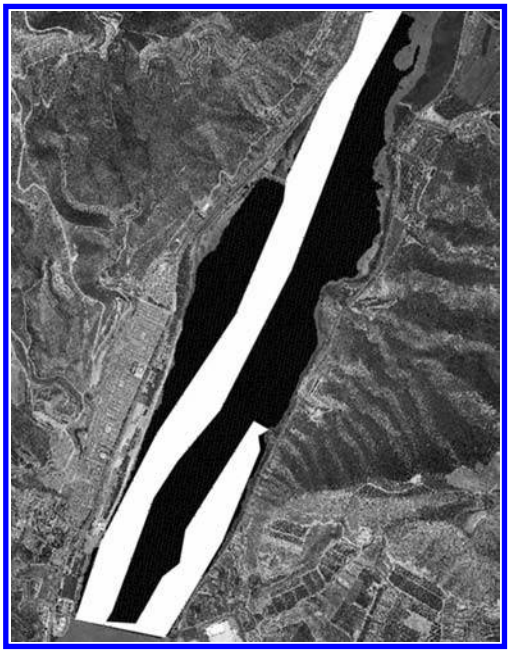

Figura 3. Ficheros de datos de campo superpuestos en la ortofotografía de la zona de estudio. En negro se define la zona que corresponde a la malla 5 × 5 m (a partir de los datos de campo de la sonda monohaz) y en blanco la zona de más detalle del canal, con una malla de 1 × 1 m (a partir de los datos obtenidos con la sonda multihaz). Agosto de 2009.

3.1.1 *Campaña batimétrica*

Para la obtención de la batimetría del fondo del embalse se realizaron medidas por perfiles transversales al río y a lo largo de todo su recorrido.

Para la obtención de la topografía del fondo se realizaron varios perfiles transversales y longitudinales al río, a lo largo de la zona de estudio, mediante ecosonda monohaz (Figura 3, en negro). Posteriormente para estudiar en detalle la zona de circulación del agua se realizó una segunda pasada con sonda multihaz, que se centró en el cauce principal del río (Figura 3, en blanco).

Finalmente se unieron los dos ficheros de las ecosondas y se depuraron los datos erróneos; obteniendo una malla regular de puntos de 5 × 5 m y de 1 × 1 m en la zona de más detalle del canal (Figura 3).

Para el tratamiento de datos se utilizaron los formatos ráster y TIN en función de los requerimientos del estudio.

3.1.2 *Unión con el modelo digital del terreno del embalse de Ribarroja existente*

Mediante ArcMap de ESRI se unió el modelo digital del embalse realizado en 2007 con el modelo digital del Segre; el resultado se puede ver en la Figura 5.

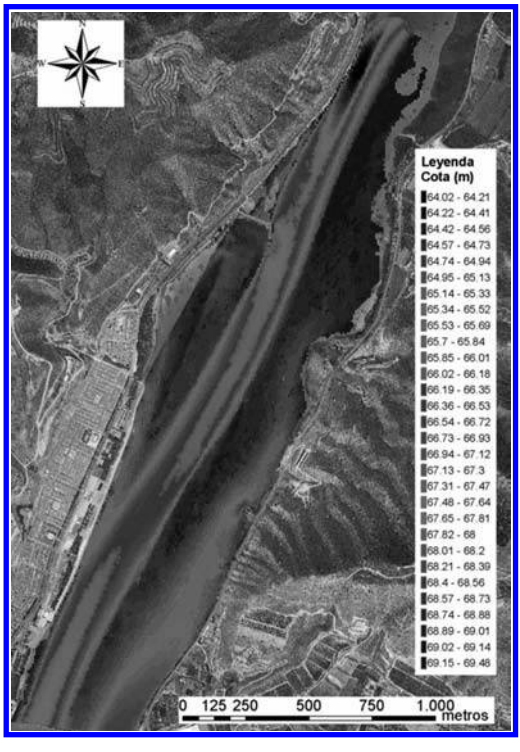

Figura 4. Detalle de la batimetría del río Segre en la zona de estudio. Cotas en metros.

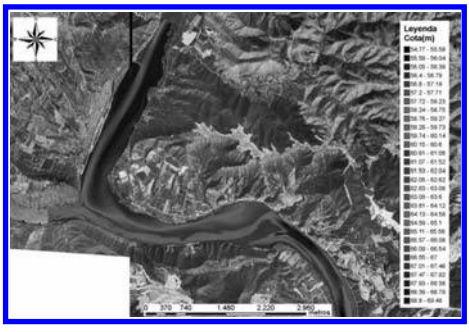

Figura 5. Unión del modelo digital del embalse de Ribarroja de Ebro (2007) con el modelo digital del Segre en en entorno de Mequinenza.

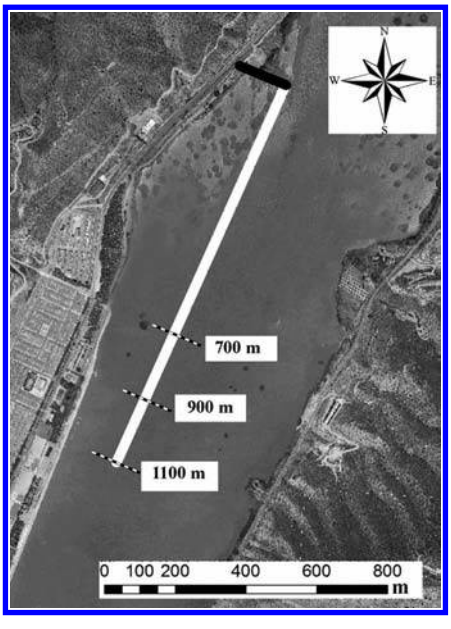

Figura 6. Longitudes de espigón simuladas. En negro se puede ver el espigón actualmente existente; en blanco se define la traza del futuro espigón a construir. Longitudes en metros.

3.2 *Casos de estudio*

Se definieron 16 casos de estudio diferentes correspondientes a diferentes combinaciones de longitudes de espigón, caudales del río Segre y cotas de embalse.

Las longitudes del espigón estudiadas fueron las tres que se muestran en la Figura 6 (700 m, 900 m y 1100 m), intentando afectar en la menor medida posible el cuace principal del río (que había quedado bien definido en la batimetría realizada, ver Figura 4). Además se consideró necesario simular también la situación actual (espigón transversal existente en la zona aguas arriba, que se puede ver pintado en negro en la Figura 6) para conocer la dinámica actual y para poder realizar las comparaciones con los casos que la modifican.

Se decidió estudiar la evolución para dos caudales diferentes: el correspondiente a un periodo de retorno de 10 años (2682 m^3/s) y otro que fuera significativamente menor aunque también importante para el río Segre, por este motivo se escogió el de 500 m^3/s.

Referente a las cotas del embalse, se propusieron dos niveles de agua: uno correspondiente a la cota media de embalse, la 69.50 m (el embalse de Ribarroja suele ser muy estable en cuanto a niveles de agua); y otro correspondiente a la cota de 68.80 m; que es el mínimo que permite asegurar un calado de 2 m (según la normativa vigente, calado necesario para el buen funcionamiento del canal de regatas) en la zona interna del espigón.

3.3 *Mallas de cálculo*

A partir del modelo digital del terreno para cada una de las geometrías se construyó una red irregular de triángulos (TIN), que se utilizó como malla de cálculo en Iber.

Para las diferentes geometrías de espigón, se consideró que la cota de la zona interior al espigón era constante de 66.80 m (propuesta en el diseño). En la Figura 7 se presenta un detalle de la malla para el espigón de 700 m.

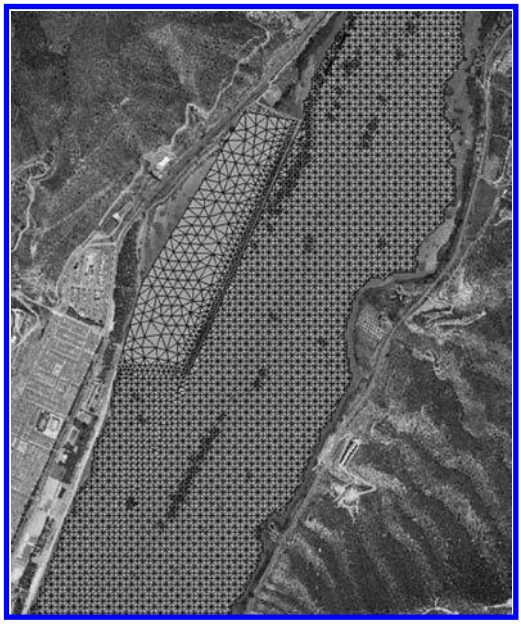

Figura 7. Detalle de la malla irregular de triángulos para el caso del espigón de 700 m.

3.4 *Resultados del modelo hidrodinámico*

En primer lugar se simuló la hidrodinámica del río Segre para cada una de las 16 combinaciones de geometría, caudal y condiciones de contorno presentadas anteriormente, y suponiendo fondo fijo, es decir, lecho no erosionable. Ello permitió conocer el detalle de calados, valores de acción hidrodinámica y campo de velocidades (Iber promedia los valores de velocidad en la profundidad).

Los resultados del cálculo hidrodinámico indican que, en la situación actual (Figura 8), existe claramente una zona de flujo preferente que coincide con la de máximo calado (coincidiendo también prácticamente con el cauce profundo del modelo digital, Figura 4).

Los calados y los valores de la velocidad varían con el caudal y el nivel de embalse, pero en todos los casos estudiados se aprecia que la presencia del espigón transversal separa el flujo de la margen derecha hacia el centro del cauce, en un tramo de unos 700 o 800 m, y aguas abajo del mismo el cauce principal vuelve a situarse muy próximo a la margen derecha como se aprecia en la Figura 8 (para el caudal de 2682 m³/s se aprecian velocidades máximas cerca del extremo del espigón de unos 3 m/s). Para el embalse a la cota 68.8 m el patrón general del flujo es muy similar a la del embalse alto (69.50 m) aunque los valores de velocidad aumentan ligeramente.

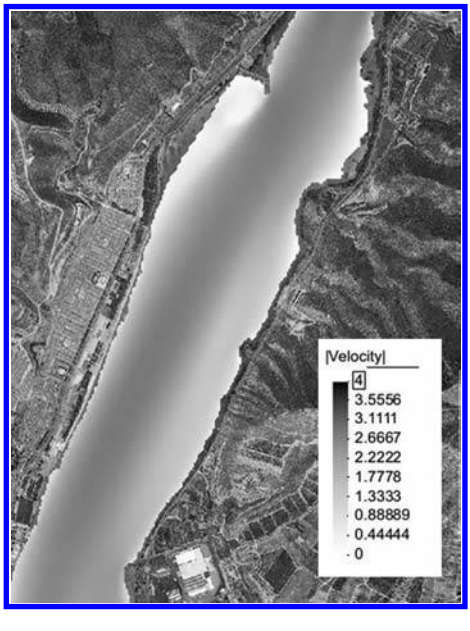

Figura 8. Detalle del campo de velocidades para la situación actual, cota de embalse a 69.50 y caudal de 2682 m³/s. Velocidades en m/s.

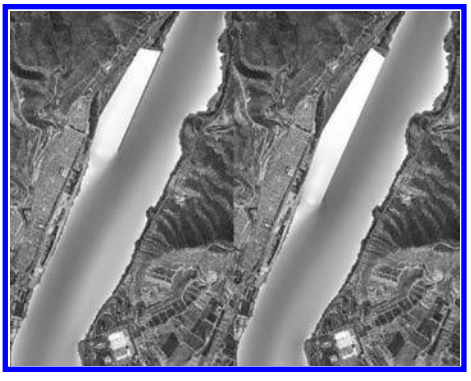

Figura 9. Detalle del campo de velocidades para los casos de longitudes de espigón de 700 m (izquierda) y 1100 m (derecha), cota de embalse a 69.50 y caudal de 2682 m³/s. Velocidades en m/s (misma escala que para la Figura 8).

En cuanto a la influencia del espigón longitudinal, se observa que la longitud de espigón que produce una menor alteración del flujo es el espigón de 700 m (Figura 9, izquierda) y en cambio, el espigón de 1100 m (Figura 9, derecha) tiene suficiente longitud para afectar al trazado cauce principal. Por otro lado, los resultados indican que a más longitud de espigón se provoca una reducción del ancho del río; y por lo tanto, las velocidades, la acción hidrodinámica (capacidad de arrastre) y los caudales específicos tienden a aumentar con la longitud del espigón.

En cuanto a la recirculación dentro del recinto que protege el espigón, el caso de 700 m deja más apertura de entrada; lo que, juntamente con el giro del cauce principal, genera que las velocidades de recirculación interior sean algo más elevadas en este caso que en los de 900 m o 1100 m de longitud de espigón.

También cabe destacar que en el caso de 700 m de longitud de espigón, el vórtice de recirculación abarca prácticamente toda la zona interna al espigón; en cambio en el caso de 900 m el vórtice se sitúa en los 650 m aguas abajo del recinto, dejando una zona de aguas más quietas junto al espigón transversal. Lo mismo ocurre para el caso de 1100 m, en que la zona de sombra es de unos 500 metros.

3.5 *Resultados del análisis morfodinámico*

Las mismas 16 combinaciones de geometrías, caudales y condiciones de contorno se estudiaron a continuación suponiendo lecho móvil.

Los resultados del análisis morfodinámico indican que las tendencias son bastante similares para todos los casos con el caudal de 500 m³/s, y también

para todos los casos con el caudal de 2682 m^3/s, pero sin embargo los patrones de erosión y sedimentación son diferentes para caudales distintos. Las distintas cotas de embalse también tienen influencia en los resultados, aunque en menor medida (a menor cota de embalse las velocidades son mayores, y por lo tanto también las erosiones).

Para un caudal de 500 m^3/s durante 24 h, y la situación actual, se producen erosiones en la zona del cauce central desde el inicio del tramo de estudio hasta aguas abajo del núcleo urbano de Mequinenza (Figura 10). El material erosionado en esta zona es depositado en parte a ambos lados del mismo cauce central y el resto aguas abajo, donde se producen sedimentaciones en todo lo ancho del cauce aunque mayoritariamente en la margen izquierda. En los resultados se puede observar la tendencia de estas avenidas con caudales moderados a colmatar el actual campo de regatas.

Para el mismo caudal de 500 m^3/s circulando 24 h, pero con espigón de 700 m (Figura 11), la tendencia general es parecida, aunque la presencia del espigón altera ente la sedimentación en las

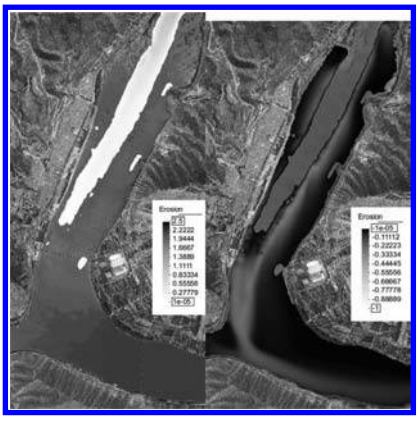

Figura 10. Erosión (izquierda) y sedimentación (derecha) en la situación actual para el caudal de 500 m^3/s (24 horas) y embalse a la cota 69.5 m. Valores en metros.

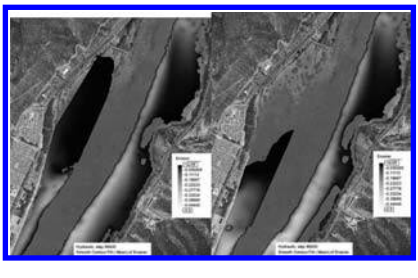

Figura 11. Erosión (izquierda) y sedimentación (derecha) en el caso de espigón de 700 m de longitud para el caudal de 500 m^3/s (24 horas) y embalse a la cota 69.5 m. Valores en metros.

proximidades de la margen derecha. Se observa que en esta margen la sedimentación disminuye de forma considerable dentro de la zona protegida por el espigón, pero aumenta considerablemente aguas abajo del mismo.

Existe cierta sedimentación en la zona inferior del área protegida, por la entrada de sedimentos por la bocana debido a las recirculaciones que se producen, pero sobre todo una tendencia a formarse una barra significativa junto a la margen derecha pero aguas abajo del espigón (detalles en la Figura 12, para los casos extremo de 700 m y 1100 m de longitud de espigón).

Para este caudal de 500 m^3/s a lo largo de 24 horas, cota de embalse 69.5 m, y el espigón de 700 m de longitud, el volumen total erosionado en la zona mostrada es aproximadamente de unos 150.000 m^3. De estos, aproximadamente una tercera parte se deposita aguas arriba del puente de Mequinenza, mientras que el resto (100.000 m^3) se depositará en lo que se puede considerar ya el cauce del río Ebro.

Los resultados del cálculo morfodinámico reafirman, en definitiva, que el espigón de 700 m es la mejor opción entre las tres longitudes de espigón estudiadas. Aun así, también muestran que el tramo de estudio está sometido a importantes cambios morfológicos, la presencia del espigón limitará estos cambios en la zona interior al recinto, pero en la parte de campo de regatas exterior al mismo se deben de prever actuaciones para el mantenimiento del mismo a largo plazo.

Para un caudal de 2682 m^3/s (10 años de periodo de retorno) durante 24 horas, la magnitud de las velocidades es considerablemente superior en toda la zona correspondiente al río Segre, de manera que la superficie sujeta a erosiones fuertes también

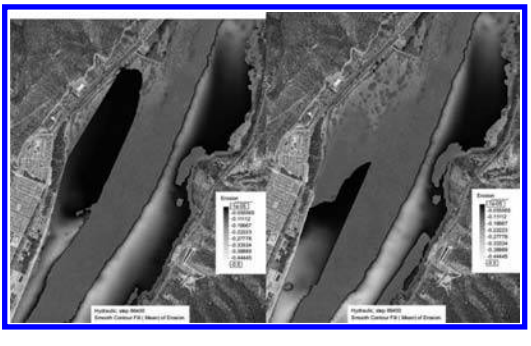

Figura 12. Detalle de la sedimentación en la parte interna del espigón (para los casos de 700 m y 1100 m de longitud), para un caudal de 500 m^3/s (24 horas) y embalse a la cota 69.5 m. Se puede observar la formación de la barra de sedimento aguas debajo de la punta del espigón en los dos casos. Valores en metros (en blanco valores cercanos a 0.5 m de deposición, en negro valores tendiendo a nulos).

es mucho mayor. En la situación actual se puede observar (Figura 13) que prácticamente toda la zona correspondiente al río Segre está sujeta a erosiones, mientras que el sedimento se deposita una vez se ha alcanzado el cauce del río Ebro y concretamente en la margen izquierda del mismo.

Como es lógico, la tendencia a formar una barra de sedimento aguas abajo del espigón se muestra también para estos caudales grandes, así como la entrada de sedimentos dentro del recinto debido al retroceso del agua. La magnitud de la barra es similar para cualquier longitud de espigón, así como el área de recinto afectada por sedimentaciones (Figura 14).

Los resultados del cálculo morfodinámico reafirman, en definitiva, que el espigón de 700 m es la mejor opción entre las tres longitudes de espigón estudiadas. Aun así, también muestran que el tramo de estudio está sometido a importantes cambios morfológicos, la presencia del espigón limitará estos cambios en la zona interior al recinto, pero en la parte de campo de regatas exterior al mismo se deben de prever actuaciones para el mantenimiento del mismo a largo plazo. Asimismo se cree de interés señalar la importante capacidad erosiva del río Segre junto a la parte externa del espigón en su extremo aguas abajo.

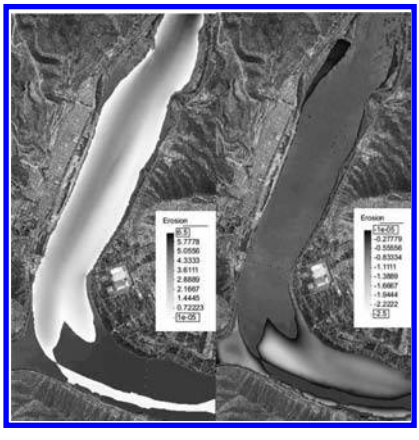

Figura 13. Erosión (izquierda) y sedimentación (derecha) en la situación actual para el caudal de 2682 m^3/s (24 horas) y embalse a la cota 69.5 m. Valores en metros.

Figura 14. Erosión (izquierda) y sedimentación (derecha) en el caso de espigón de 700 m de longitud para el caudal de 2682 m^3/s (24 horas) y embalse a la cota 69.5 m. Valores en metros.

3.6 *Permeabilidad del espigón*

Para evitar en la medida de lo posible las aguas estancadas dentro del recinto del campo de regatas, también se estudió el efecto de practicar distintas aperturas en el espigón longitudinal para permitir el trasvase de caudal entre el río y el campo de regatas.

Inicialmente se estudiaron tres casos con una única apertura en el extremo superior del espigón longitudinal (de 10, 20 y 30 m), y un caudal por el Segre de 100 m^3/s (en la Figura 15 se pueden ver los detalles del campo de velocidades de los casos de 10 y 30 m de apertura).

Para ellos se observa que las magnitudes de las velocidades son parecidas pero los patrones de circulación difieren entre la apertura de 10 m y las otras dos. Se puede observar que para la primera,

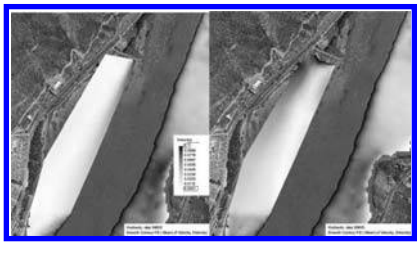

Figura 15. Detalle del campo de velocidades para un caudal de 100 m^3/s en el Segre. Aperturas de 10 m (izquierda) y 30 m (derecha). Valores en m/s (se ha exagerado la escala de colores para destacar las magnitudes en la zona interna del espigón).

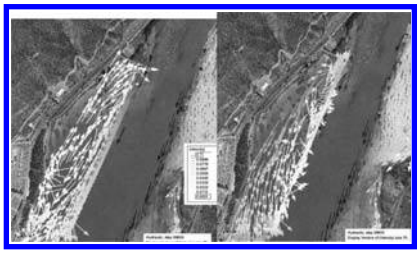

Figura 16. Detalle del campo de vectores de velocidad para un caudal de 100 m^3/s en el Segre. Aperturas de 10 m (izquierda) y 30 m (derecha).

el agua que entra tiene dirección preferente hacia aguas abajo en las inmediaciones de la apertura generando un vórtice de circulación en sentido horario. En cambio, en los casos de 20 y 30 m de apertura, se puede ver que los vectores de velocidad predominantes junto a la apertura se dirigen perpendicularmente a la dirección del flujo principal, provocando una circulación en sentido antihorario. En los casos de 20 y 30 m de apertura las magnitudes de la velocidad en la entrada son superiores, y también la afectación a la margen derecha del río.

También se estudiaron casos en que se consideaba más de una apertura, pero las aperturas adicionales influyen poco en la magnitud de la velocidad aunque si influyen en el patrón de recirculación, dando lugar a un movimiento más desordenado junto a la cara interna del espigón.

4 CONLUSIONES

Del estudio en modelo numérico de la hidrodinámica del río Segre en el entorno de Mequinenza, se desprenden las siguientes conclusiones:

Para la delimitación del recinto del campo de regatas, se considera que la solución más adecuada entre las estudiadas es la de longitud más corta, debido a que afecta en menor medida al cauce principal del río, y a su vez provoca un menor impacto sobre la margen izquierda. Mayores longitudes de espigón provocarían un cambio significativo en la capacidad erosiva en el entorno del extremo aguas abajo del espigón, y también en el resto del cauce.

El análisis con fondo móvil permite conocer la tendencia evolutiva del cauce ante el paso de una avenida. Para el caudal de 500 m^3/s la zona de erosión queda limitada al tramo aguas arriba del puente de Mequinenza. Para el caudal de 2682 m^3/s (correspondiente al periodo de retorno 10 años) se observa una tendencia erosiva en todo el cauce del río Segre, mientras que el sedimento se deposita en el embalse de Ribarroja después de la confluencia con el río Ebro.

Un espigón longitudinal como el propuesto limitará en gran medida la deposición de sedimento dentro del recinto encerrado por el mismo. Sin embargo propiciará la formación de una barra junto a la margen derecha, aguas abajo del extremo del espigón, y también en la zona próxima a la apertura se pueden observar deposiciones tras el paso de una avenida.

Para conseguir un mínimo flujo de agua en el recinto, se sugiere practicar una apertura en el extremo aguas arriba del espigón longitudinal, controlada por compuertas. El patrón de flujo cambia de forma significativa entre un ancho de apertura de 10 m y uno de 20 m. Para aperturas mayores, o más aperturas, el patrón es similar aunque cambia la magnitud del caudal circulante y por tanto las velocidades.

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Global analysis of the sedimentation volume on Portuguese reservoirs

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ABSTRACT: This paper aims to approach the process of sedimentation in reservoirs, namely, trying to estimate the sediment volume deposited in the main Portuguese ones. The changes in fluvial sediment transport induced by the construction of dams on major Portuguese rivers are described. Based on the values of dead volumes, the current situation of 166 Portuguese reservoirs is analyzed in order to obtain an order of magnitude of the sediments available and pre-select the reservoirs that could potentially integrate a project for artificial sand nourishment of beaches. This work was carried out as a starting point, in order to implement a national plan for the exploitation of sediments considered to be an "added—value" to the economy and that are today, in general unexploited.

1 INTRODUCTION

The coastal erosion compromises the future of the littoral populations and so it is of major interest to search for a solution that allows controlling that effect on the best way possible.

Although it represents an extremely important role in the energy production, the construction of dams in the rivers' networks in Portugal triggered the upstream retention of a volume of sediments that would nourish the beaches, contributing for the aggravation of the coastal erosion and for the reduction of the volume of water storage available for the regulation of the rivers.

2 PROCESSES OF SEDIMENTATION IN A RESERVOIR

The construction of a dam modifies the sediment transport conditions representing, usually, a breach in the repetition of the usual sequence of erosion, transport and deposition, with the formation of deltas in the upstream border of the reservoir, silting of the mentioned border and erosion following the restitution, Figure 1, Lysne et al. (2003).

So, when designing a reservoir it is mandatory to save some space for the sediments deposition, the dead volume, which is defined according to the scheme of solids' transportation of the river and of the probable working life of the structure, Lencastre and Franco (2006).

The deposition of the biggest part of the sediments occurs in the upstream of the reservoir, in the useful storage volume and not in the dead volume (volume below the lowest level of the water intake).

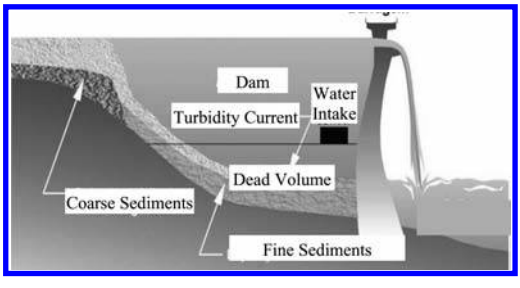

Figure 1. Typical pattern of sediments' deposition in reservoirs. (http://www.dha.lnec.pt/nre/english/projects/sedim_eng.html)

After the deposition, the coarse materials show a much bigger stability, maintaining its characteristics in time and with a more difficult aftermost motion. On the contrary, the soft sediment deposits show an evolution of its properties suffering a compression through time, allowing the superficial layers to be placed in suspension much easier.

3 TREATMENT METHODOLOGIES

The sediments on the bottom of the reservoirs are known as preferential placement of organic deposits and mineral materials, namely, nutrients, heavy metals and bacteria, considered in its whole as pollutant sources.

Aiming the artificial beach nourishment it will be more important to evaluate the concentration of heavy metals in the sediments than the content in organic matter once this can actually be positive when the sand is applied to dune ridges making

conditions to the development of vegetation. The process of sediment treatment removed from reservoirs, in order to prepare them for its several purposes, may be divided in two different stages: granularity distribution and decontamination.

The granularity distribution is usually obtained through bolting. For beach nourishment the required fraction corresponds to the sand fraction (dimensions between 0.075 mm, #200 and 4.75 mm, #4), the granularity shall be the closest as possible to the granularity of the beach to nourish. The beaches nourished with higher granularities will be more stable but steeper, inducing new profiles with possible implications at the comfort and beach safety level. Beaches nourished with smaller granularities are potentially more instable and so the volumes placed will tend to be sooner washed away by the sea.

It has been accepted that polluted particles are concentrated in the softer fraction of the sediments, Colandini (1997), Lee et al. (1997), Zanders (2005), referred by Pétavy et al. (2008). Nevertheless, recent studies, show that the metals don't establish themselves to a specific fraction of sediments, Durand (2003), Clozel et al. (2007), referred by Pétavy et al. (2008).

Therefore, it will be convenient to always check the need of decontamination through chemical analysis before proceeding to the transportation of the sediments to apply on the beaches. The need for sand decontamination means expensive costs in a project of sediments reuse to artificial beach nourishment which can actually lead to the impracticability of the project and so it should be well evaluated. Nevertheless, normally, this process of decontamination may be dismissed in what concerns the coarser fraction.

4 APPROACH AND ANALYSIS OF THE PORTUGUESE CASE

4.1 *Introduction*

The natural erosion phenomenon has turned into a problem of growing intensity due to the human intervention on coastal areas, once that the low altitude areas that were naturally capable of, adapting themselves to the variations of the average level of the sea, storms and tides have lost nowadays such ability due to the occupation of the coast line with roads, urbanizations, playgrounds, industrial areas, among other infra-structures.

Being the construction of dams pointed as one of the main causes for the reduction of sediment transportation in the rivers, it was estimated that such reduction ascends to 80% in Portugal, Fonseca (2002). In 2008, the investment in coastal defense works in Portugal was 7 579 000€, Maia (2009).

A project of reuse of deposited sediments in reservoirs to artificial nourishment of beaches may involve high costs, nevertheless, in the evaluation of the viability of such projects it will be necessary to mind the dimension of the required investments either for the implementation of alternative solutions as well as for its maintenance once the repair and adaptation of the defense works have been a great part of such investments.

4.2 *Approach to the Portuguese case*

In Portugal big forest fires occur quite often during the summer devastating completely the vegetation in areas with concerning dimensions allowing the increase of the affluence of sediments to watercourses.

The main direction of sediment transport in the Atlantic coast is from north to south. In order to give an idea on the capacity of transport of the sea some references indicate that it is near 1 000 000 m^3/year from the Douro's river mouth to Nazaré, Veloso Gomes et al. (2006b).

In 1930, previously to the construction of the dams the affluence of sediments to the Portuguese north shore was approximately of 2 000 000 m^3/year, with the River Douro itself assuring 90% of this value (National Water Plan, 2002). Nowadays, the River Douro only contributes with a small amount estimated in 250 000 m^3/year as a result of the dam construction that reduced the scour speed and subsequent ability to erode the river bed.

Aware of this, there is also a meaningful amount of sand extracted for building industry use dredged from the river's bed in several places and in amounts beyond what it would be sustainable, Veloso Gomes et al. (2006a).

The mean rates of sedimentation calculated in the period between 1983 and 1984, in the Tejo's estuary, point to values of 1,1 and 1,5 cm/year. During the construction of Vasco da Gama's Bridge about a million of m^3 of sediments were dredged from the Tejo River, that were, in some way, contaminated, PNA (2002). In the Guadiana river one of the consequences of the flow reduction, induced by the hydrological loss and regularization of the huge storage installed (70,4% of the basin dominated in 1990), is the decrease of the sediments supply, also to the adjacent shore areas, PNA (2002).

In Portugal, the importance of the evaluation of the reservoirs' dredging is recognized as well as its consequences and the need of the existence of a permanent group for its analysis.

Since 1996, the Water Institute (INAG) has been restructuring the monitoring networks of hydraulic resources including the sediment network. The new sediment network includes stations

in watercourses where it will be performed samplings of solid flows and bottom granularities and stations in reservoirs where bathymetric surveys will be performed. As the network is still in implementation the results are not yet known.

4.3 *Balance and analysis of the current sedimentation in the main Portuguese reservoirs*

Although the efford being made in the way of bridging the lack of information regarding the sediments transport in Portugal, the amount of deposited sediments in the reservoirs remains, nowadays, unknown and consequently so does the actual capability of the Portuguese reservoirs.

There is a lack of information regarding sedimentation and so the data that are part of the basis of this study comes from the inventory of Portuguese dams made by INAG. The dams in this inventory generally obey to the criteria of having more than 15 m high or having more than a million m^3 of total storage capacity.

Mainly through the values presented by INAG to the dead-volumes and using information regarding the siltation rate (Lencastre and Franco, 2006) an attempt was made to estimate values that, in a way, can show the current situation of the sedimentation in those Portuguese dams. In the calculation the design dead volume of each dam was considered, defined during the project phase, as being the real volume of actual sedimentation verified in the reservoir with 50 or more years of existence:

$$Volume_{sedimentation} = Volume_{dead} \quad (1)$$

and for dams under 50 years of existence it was defined that the sedimentation of the reservoir would be directly proportional to the age of the dam:

$$Volume_{sedimentation} = \frac{Volume_{dead}}{50} \cdot age_{dam} \quad (2)$$

With the purpose of being able to bridge the lack of data, the procedure was similar to the suggestion for the volume balance of the deposited sediments volume referred in ICOLD (1989) in which it is established that a empirical relation between the basin area and the volume of deposited sediments. Such relation was defined separately to the hydrographic areas of North, Center and South in an attempt of isolating the cases with similar conditions of erosion and transportation, Figures 2, 3 and 4.

Although these relationships don't present high correlation degrees, it was considered that they are capable of supplying sediment volumes with enough vicinity. It is necessary to highlight that the base assumptions were adopted in order to obtain a order of magnitude and that once in the presence of more accurate data such assumptions will no longer make any sense.

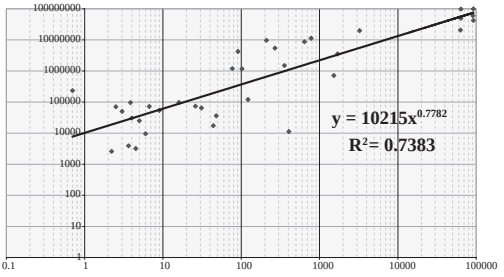

Figure 2. Relationship between the deposited sediments volume (yy-m^3) and the hydrographic basin area (xx-km^2) in the North region of Portugal.

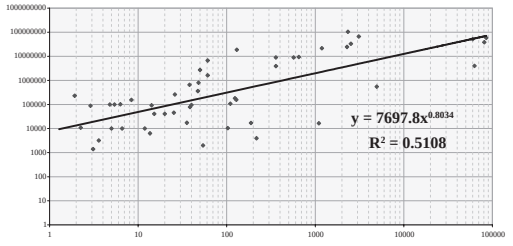

Figure 3. Relationship between the deposited sediments volume (yy-m^3) and the hydrographic basin area (xx-km^2) in the Center region of Portugal.

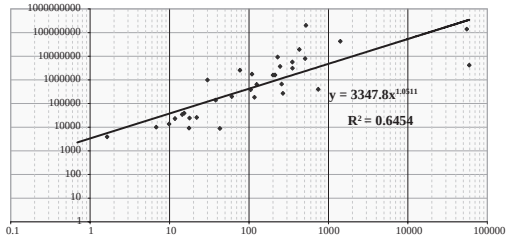

Figure 4. Relationship between the deposited sediments volume (yy-m^3) and the hydrographic basin area (xx-km^2) in the South region of Portugal.

The balance of INAG consists in 166 dams and it was possible to gather enough information to estimate, for the already mentioned purposes, the volume of deposited sediments in 151 of the 166 dams.

It is estimated that these 151 dams represent a global volume of deposited sediments of 1568 millions of m^3 and a total gross capacity of 12546 millions of m^3 which represents that 12.5%

of the total gross capacity for the water storage is, today, occupied by sediments.

Of the 151 dams with estimated sedimentation volumes, about 66% have a volume of sedimentation below 10% of its gross storage capacity, 28% are between 10 and 50% and only 6% are above 50% of its gross storage capacity.

Using the ArcGis software the coordinates of the dams were referred in the Geographic Information System which allowed the analysis of the positioning of the 166 reservoirs regarding the Portuguese shore, Figure 5. Currently, Portugal is divided in 5 Hydrographic Regions that are also defined in Figure 5.

Such division seemed as well the most adequate once the observation of the map allows a faster reading of the distribution of the reservoirs through the Hydrographic Regions that have jurisdiction over the correspondent coastal areas, suggesting a way of managing the entire process of reuse of the sediments.

Of the 166 reservoirs those that would be more interesting for a project of reuse of the sediments to beach nourishment were selected. The selection criteria was the following:

- Reservoirs with over 0.2 hm^3 of sediments and located less than 50 km to the shore;
- Reservoirs located more than 50 km and less than 100 km to the shore and with more than 0.5 hm^3 of sediments.

These criteria are similar to those of a study made in Spain with the same purpose, Sanchez (2008).

The selection of reservoirs represents a preselection that will require the continuous investigation before it can be decided conclusively about its insertion in a project of reuse of the sediments to beach nourishment.

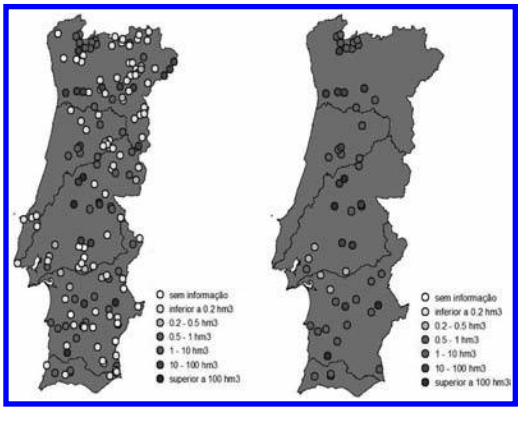

Figure 5. Localization of the 166 reservoirs considered (left), and selected (right), INAG.

Of the 166 reservoirs analyzed, 53 were selected representing in its whole 1242 millions of m^3 of sediments and 10968 millions of m^3 of gross capacity for water storage. Of all reservoirs selected, 11 are placed to less than 15 km of the shore, 17 between 25 an 50 km, 10 between 50 and 75 km an 15 between 75 and 100 km representing, respectively, 65, 484, 345 and 347 millions of m^3 available, Figure 6.

Regarding the hydrographic regions the selection made regards to 269 millions of m^3 in the North Hydrographic Region, 84 millions of m^3 in the Center and 482 millions in Tejo's Hydrographic Region, 394 millions of m^3 in the Alentejo and 13 millions in Algarve.

The Hydrographic Regions of Tejo and Alentejo are the ones that present the biggest amount of sediments available maybe due to the existence of regularizations of major dimensions. The hydrographic regions of Algarve and Center present a smaller amount of available sediments even because they are also the regions with less reservoirs analyzed.

5 FINAL REMARKS

Currently, in Portugal, there is no strategy for the management of sediments, nor is recognized, along with the problem of coastal erosion, the existence of the sedimentation problem in reservoirs, because it appears with less intensity, representing nowadays a smaller concern to the society in general, but that, in case of no intervention it will be a problem to face in the future.

So, as soon as we plan a strategy for intervention, smaller will be the effort required to implement it. Also there is no single solution for this problem as each reservoir has its specific characteristics.

The transport is the conditioning factor of a project for reuse of the sediments for beach nourishment. The transport by land is expensive and has significant negative impacts that rise with the distance. This requires the search for alternative solutions. The transport by river can be a solution in the navigable sections of the main rivers.

In a cost-benefit balance it should be take to account the benefits regarding not only the reuse of the sediments but also the reduction of the associated costs to the coastal defense works, that represent high costs, and the maximization of the profit taken for the dams, that, by all the major impacts that represent justify the maintenance works that allow its functions.

For all this, it is important to recognize that the current investment in the revitalization of the sediment networks constitute an excellent opportunity to the definition of a strategy for the management of the sediments that can be placed in practice in

an effective way, regarding an exhaustive use, as possible, of the sediments, that are a major gain to the country economy.

Such projects require technical studies and financial support and also political will. It is mandatory the coordination of all the involved parts: water companies, industrial promoters, coastal and environmental authorities and the transparency within the users: habitants, fishermen and landlords of the reservoir surroundings.

This work was carried out as a starting point, in order to implement a national plan for the exploitation of sediments considered to be an important gain to the economy and that are today, in general unexploited.

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Testing of reservoir capacity and sedimentation of Piva lake

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ABSTRACT: This paper presents the methodology of bathymetric survey made in extremely harsh field conditions such as Piva Lake and Mratinje dam, by using a combination of satellite and terrestrial surveying methods. The modern approach of data processing and spatial analysis of the data was applied. In order to reach relevant conclusions regarding the state of the reservoir and propose future activities, an additional geo-statistical analysis was performed for the purpose of determining the extent of sedimentation and possible decrease of reservoir storage capacity.

1 INTRODUCTION

1.1 *Location and main characteristics of reservoir*

The Piva hydro-reservoir was formed by constructing Mratinje dam. It is located in the northwest of the Republic of Montenegro. Mratinje dam is an arch dam with the construction height of 220 m, which makes it one of the highest dams in Europe. The main characteristics of the reservoir are as follows: storage capacity of around 825 million cubic meters, water area of 15.5 km^2, length of around 60 km, including bays, and maximum depth of around 180 m. It is a specific hydro-geological system with very deep canyons of the Piva and Komarnica rivers and inaccessible surrounding terrain. Comprehensive investigations of the reservoir storage capacity and assessments of sedimentation were carried out for the first time after 35 years of operation.

1.2 *Type of works*

Subject of works under the investor requirements are geodetic survey related to capacity testing and reservoir sedimentation of HPP Piva. Works include a geodetic survey to defined height point 675.25 m, in length of about 60 kilometers with all bays.

The geodetic survey is realized by a combination of tachymetric surveying of the lake shores (terrain above water) with bathymetric survey of the bottom of the lake, below the surface. For definition of the geodetic survey datum, a unified geodetic network was previously established, materialized with geodetic concrete beacons.

Based on the geodetic determination of Y, X and H coordinates of accumulation crossections endpoints and mass points, the digital terrain model was created according to which necessary analysis and calculations were made.

2 PREPARATORY WORKS

2.1 *Geodetic network*

Field recognition of the existing geodetic network was carried out, followed with design and realization of the geodetic base. An established geodetic network is a necessary precondition for the implementation of the survey. The network is materialized with concrete a beacon in the state coordinate system and in geodetic datum of the Republic of Montenegro. The network includes the wide area of HPP Piva.

The adopted concept of geodetic network includes the realization of the basic 2D and 1D and secondary 2D and 1D geodetic network. Global positioning system (GPS) technology and terrestrial conventional methods of measurement were used.

2.2 *Defining the optimal survey method*

Volume calculation based on the measurements along the profiles is a method used in the past in accordance with technology which was available then. Modern technologies and techniques significantly raised the standards in terms of accuracy.

Regarding volume reservoir calculation, measurement systems and software for data processing provide much more reliable and accurate results.

Analyzing all the possibilities i.e. methods of survey and processing of recorded data, it can be concluded that the accumulation should be

Figure 1. Spatial distribution of surveyed points.

surveyed with integrated global positioning system (GPS) or total station and ehosounder, along and between crossections using mass points method and structural lines of the underwater terrain (line of greatest depth and underwater terrain brake lines). Figure 1 shows the placement of points recorded by these methods on a part of the reservoir.

Reservoir volume, volume curve and other data should be calculated using Digital Terrain Model (DTM).

3 GEODETIC-BATHYMETRIC WORKS

3.1 *Crossections' marking and stabilization*

Before the beginning of the bathymetric survey, 327 crossections endpoints of the lake were stabilized. Crossections endpoints were stabilized with concrete beacons with iron peg, drilled in rock to 80–100 mm depth.

Crossections endpoints are placed above the height 676.25 m. in places that are easily observed from the surface of the lake.

All crossection endpoints are numbered in a unified system. During the stabilization of endpoint crossection, required measurements, sketching and photo shooting was carried out, enabling the creation of a crossection endpoints position description.

Coordinates of crossection endpoints are determined from geodetic network points, using the terrestrial survey method. Control was carried out based on multiple measurements. Coordinates were obtained using an arithmetic mean of results that were within the allowable tolerances.

3.2 *Survey of dry reservoir above the current water level*

Temporary dry reservoir terrain, above the lake's surface up to the height of 676.00 m was surveyed. Total stations were used for the survey: Leica with a laser range of 1000 m and Trimble with a laser range of 400 m. This technology allows the recording of inaccessible parts of terrain. Data (coordinates) of recorded terrestrial reservoir terrain are associated with the data (coordinates) of the underwater reservoir terrain survey, thus making it possible to develop a reservoir model.

3.3 *Bathymetric survey of the accumulation*

Proper equipment and software for bathymetric survey and data processing was used, such are: specialized boat SM-26A, GPS equipment with base station and rover associated with sonar on the boat (Trimble 5700), geodetic echo sounder (ATLAS DESO 300 with a probe of 210 kHz), built in boat bed and software for hydrographic measurements and data processing (Trimble HYDROpro). Combining these devices and software provides a unique integrated system for measuring point position and depth in the reservoirs.

Data on the depth (echo sounder) and position (dual frequency GPS receivers) are integrated and linked to the computer where it's special software package for Hydrographic measurements (Trimble HYDROpro) synchronizes them by time, so that the position of the echo sounder probe (in the same vertical line with the GPS antenna) is constantly known in real time (real time kinematic GPS positioning method). All the above equipment is mounted on the boat SM-26A.

Usage of differential GPS method for bathymetric survey was planned and conducted in the period when the GPS satellites were in the most favorable position to obtain an accurate position. As a basis for the development of this plan, GPS satellite ephemerides were used and the approximate location of the survey with an elevation mask. In this survey, the base GPS station was always located at the points on the control geodetic network on the surrounding peaks in order to have the best possible reception of radio signal. The main problem was receiving a signal from satellites on the rover that was on the boat, which was resolved by a survey plan for most favorable time.

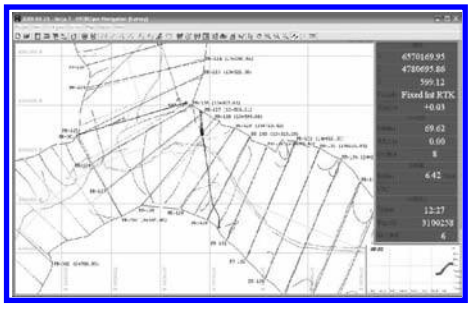

Figure 2. The appearance of the user interface software HYDROpro Trimble Navigation.

In addition to these crossections, characteristic picks or pits of underwater terrain of reservoir HPP Piva were surveyed. The survey was carried out from the middle of the lake to the coast allowing visual data control and monitoring on the computer. Part of the reservoir, just above the dam, 400 m in the length, was surveyed by a square network located 20 m from each other in order to view the complete morphology of the lake (defining the position of pre-dam) and the reservoir dam itself. Additionally, recording was carried out along the main stream in one to three passes, thus creating a reliable survey of the underwater terrain morphology. Greater point's density and better control of data was obtained from independent surveys along and transverse the reservoir bottom. The height of water was read on the boat, and controlled by the height water measured on official staffs on the dam.

The bathymetric survey team consisted of the following: steersman who was responsible for boat and navigation and managing the boat sails by predefined course; and a geodetic expert who managed the survey system, controlling instruments and data. Measurement control consisted in controlling the quality of base-rover radio communications and the accuracy of position which directly depends on the number of satellites whose signals are received by the rover. Also, the geodetic expert "guided" the boat enabling detailed survey of the underwater terrain morphology between profiles. At the end of the day sequence analysis was performed on the transferred data to the computer in order to establish accuracy and density of measured points and reject all pseudo-solutions which are obtained as a reflection of the cluster of fish, submerged wood, etc. Boat speed was adapted to the characteristics of the embedded systems for bathymetric survey and did not exceed 8 km/h. At this speed maximum accuracy of the GPS antenna position is +/–10 cm.

In the canyon Komarnica, where the GPS signal reception was poor due to the high and steep cliffs where the position of RTK GPS method was unreliable or impossible, the position of the rover antenna on the boat was determined by terrestrial geodetic methods. A 360° type prism is built under the rover's antenna, which enables the determining of the rover antenna's position from the geodetic network control points using robotic total stations. For positions determined in this way, the depth was determined using echo sounder. This method was also used to control locations where the signal from the minimum necessary number of GPS satellites was received.

3.3.1 *Quality control of the bathimetric measurements*

Control consisted of mutual comparison of points and profiles obtained from independent measurements. Control of measurements were carried out by comparing the data for the same site that was taken from two different passes (survey line), as shown on Figure 3.

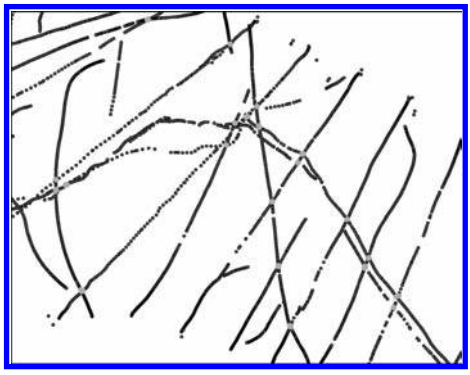

Figure 3. Independent bathymetric survey control.

In addition to "random" intersections of survey directions which are a consequence of taking a detailed survey, controls were planned and carried out in parts of the reservoir, which were recorded the previous day.

The control measurements result fits into the declared accuracy system for bathymetric survey.

4 SURVEY DATA PROCESSING

Processing the collected data included the following activities: integration of data collected by various geodetic methods, filtering and customizing data, creating Digital Terrain Model (DTM) of the lake bed, calculating the Piva Lake volume and creation of profiles layouts.

4.1 *Integration of data collected by various geodetic methods*

For the purpose of three-dimensional modeling of Piva lake accumulation, it is necessary to integrate all data collected with different geodetic methods. With that aim a single CAD file was formed, which contained recorded and previously filtered data from the bathymetric survey and survey of the dry part of the lake bed. The total number of pre-filtered points was 343,124.

4.2 *Filtering and customizing data*

This phase precedes the development of DMT. It is a necessary prerequisite for obtaining a correct terrain model, for that reason the greatest attention is dedicated to this phase.

Data filtering removes the remaining noise (errors) in the data. Interactive manual filtering was used, which means that several iterations of data analysis and building temporally terrain models

will remove the remaining noise. This method requires more time but it is significantly more reliable than automatic filtering. Figure 4 shows the characteristic appearance of the noise.

Due to known limitations in performing bathymetric surveys on such extremely difficult terrain configuration, especially in the canyon of the river Komarnica, it was necessary to manually define the structural lines of terrain in situations where the density of recorded points and their position do not provide correct modeling. Continuous break lines were also mapped by connecting points of the recorded line of the lake level and points of the border line of terrain. The aim of this procedure is to raise the quality of the digital terrain model.

The total number of points that are included in the formation of terrain model amounts to 335,714, which means that an additional 7409 points (or 2%) are removed. A total of 7847 terrain lines were mapped.

Customizing data also meant bringing them in a format suitable for further analysis and required computations.

4.3 *Creating digital terrain model (DTM) of the reservoir*

A Triangulated Irregular Network of points (TIN) was created, based on a given set of 3D coordinates. The production of the final digital terrain model is achieved by method of interpolation. A raster terrain model was produced in resolution 5×5 m. It consists of a network of pixels located in certain space each assigned with a value that represents the height above sea level. This format allows further calculations.

Preliminary analysis of data established grid size 5×5 m as optimum. For this purpose several options were analyzed, based on the sample on location of Pivsko oko—Đatl bridge:

- TIN based on the filtered 3D points data.
- TIN based on raster grid 3×3 m,
- TIN based on raster grid 5×5 m,
- TIN based on the raster grid 10×10 m

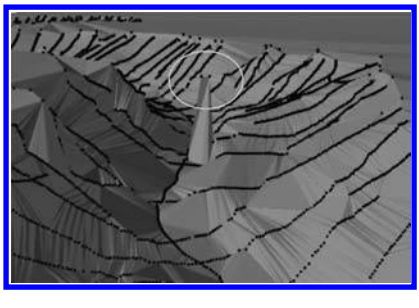

Figure 4. Filtering data.

The first analysis included calculations concerning the volume of the selected sample based on four models of the terrain on 10 different levels of terrain. Table 1 presents relevant parameters of the conducted analysis.

From the previous table it is obvious that the optimal variant of the raster grid size is 5×5 m, since it is least likely to deviate from the volume value and profile position of the selected sample.

Total number of points generated on the basis of raster grid size 5×5 m is 643,677.

4.4 *Piva lake volume curve calculation*

Volume and volume curve of the reservoir was calculated according to the adopted optimal terrain model. This means that values of the lake volume are calculated on selected field heights. Max calculation height was 676.25 m and min 500.00 m. Resolution was 1 cm from height 595 m to height 767.25 m. From height 595 m to height 500 m densities amounted to 25 cm. Water level 595 m is the minimum operational level of dam system.

2D and 3D surfaces on selected heights were additionally calculated. Surface 2D refers to the surface of the water surface level, and 3D surface is the total area of the terrain below projected water level.

Volume curve and a table of the main indicators of the reservoir's potential are delivered to the Investor in digital form. Table 2. shows a sample of the table, since the calculations were performed on a total of 8027 projected elevations in resolution of 1 cm, which represents the value according to which lake height readings are frequently conducted by the dam's staff.

4.5 *Creation of profiles layouts*

At the dam location of the reservoir, profiles layouts are made on projected crossstations in 1: 1000/1000 scale. In canyon Komarnica where the width and depth of the river bed is minimum, for better visibility, profiles were mapped in the 1:500/500.

5 ASSESSMENT OF THE RESERVOIR STATE

Construction of the "Mratinje" dam and the formation of Piva Lake disturbed the dynamic equilibrium that existed in Piva river stream. Water speed is reduced, and therefore its sedimentation as well. In reservoirs of this type, sedimentation is reduced from the mouth of tributaries along the reservoir and thus sediment coarse fraction is retained on the upstream end of the reservoir forming delta.

The main tributaries of Piva Lake are: Komarnica river, Vrbnica river and Mratinje stream.

Table 1. Finding a optimal variant of the DTM raster grid size.

H [m]	Volume TinData [*10⁶ m³]	RasterTin3 m [*10⁶ m³]	RasterTin5 m [*10⁶ m³]	RasterTin10 m [*10⁶ m³]
675	71.282	71.298	71.283	71.215
670	62.820	62.835	62.820	62.751
665	54.817	54.831	54.816	54.748
660	47.111	47.125	47.109	47.040
655	39.910	39.922	39.905	39.836
650	33.274	33.282	33.265	33.195
645	27.213	27.217	27.200	27.129
640	21.707	21.707	21.689	21.619
635	16.749	16.745	16.728	16.661
630	12.349	12.344	12.328	12.263
Sum	387.231	387.305	387.143	386.457
The most probable sum value	387.034			
Differences from the most likely sum value	0.197	0.271	**0.109**	–0.577

Table 2. Relevant indicators of reservoir state in volume, 2D surface and 3D surface sense.

The water level [m]	Volume [million m³]	Effective volume [million m³]	3D surface [km²]	2D surface [km²]
675.25	824.557	746.144	18,8941	15,4143
675.24	824.403	745.990	18,8924	15,4130
675.23	824.248	745.836	18,8906	15,4116
675.22	824.094	745.682	18,8889	15,4103
675.21	823.940	745.527	18,8871	15,4090
675.20	823.786	745.373	18,8853	15,4076
674.00	805.394	726.981	18,6647	15,2457
670.00	745.492	667.080	17,8962	14,6927
665.00	673.936	595.524	16,8650	13,9049
650.00	484.070	405.657	13,7686	11,4399
630.00	286.090	207.677	10,0161	8,4051
600.00	97.918	19.505	5,0244	4,2149
595.00	78.413	0.000	4,3025	3,5977

The river Komarnica flows into Piva Lake about 40 km upstream from the dam. Komarnica flows through low erosive soil and therefore does not yield large quantities of sediment. At the top of the reservoir in canyon Komarnica during the descending of water level, sedimentation is observed and it occurs in the form of sandbanks. In comparison with the capacity of the reservoir and the time period in which it was created, the amount of sediment can be considered insignificant.

Vrbnica River flows into the Piva Lake at the top of Plužine Bay. The river is created by the merging of several smaller rivers and streams that occur on the slopes of Lebršnik. Alignment profile is steeper than Komarnica's profile. It flows through lower erosive land, and in times of great rainfalls and snow melting it takes on the characteristics of a flood. When the water level in the lake is lower, sedimentation is clearly observed at the delta of Vrbnica. The river curves through. As Vrbnica is a small river, it means that the amount of sediment that it brings is not great. Considering the time of sedimentation deposition in comparison with accumulation volume, it can be considered insignificant. However, in order to fully protect the reservoir, the construction of the barrier on Vrbnica could be considered in order to settle the flow.

Mratinje creek has small capacity, a small basin and does not bring significant amounts of sediment, which is also observed on the location where it flows into the lake.

Part of the reservoir upstream from the dam in the length of about 400 m is surveyed in detail.

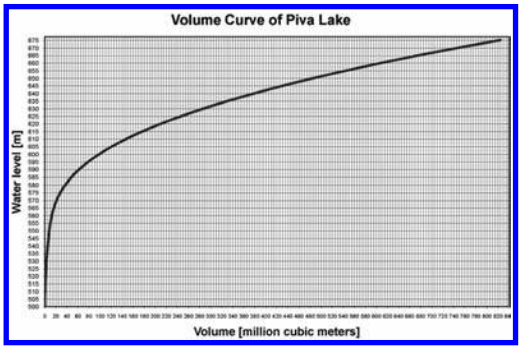

Figure 5. Piva Lake volume curve.

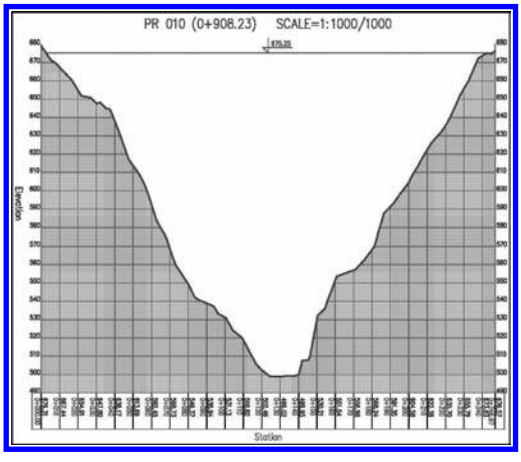

Figure 6. One of the characteristic profiles layout.

Based on this collected data, some objects were "discovered", such as a temporary dam and entrance building. Comparing the heights obtained using bathymetric survey with heights from the dam project it can be concluded that this part of the accumulation does not have significant sedimentation, but a thin layer of material.

To determine possible sedimentation of the whole lake's reservoir, it is necessary to compare results of the current survey with results of the previous one or with the so-called zero situation. As this is the first bathymetric reservoir survey, an attempt was made to make a comparison with the data which was at disposal for the terrain that was sunk by reservoir filling. It is represented by the topographical map in 1:25000 scale with equidistance of 10 m.

In order to determine possible accumulation sedimentation, either with drift or coast landslides, 3D digital models were made from all available surfaces.

Digital 3D model based on the topographic layout in 1:200 scale for the temporary dam's location. This model is compared with the bathymetric model. It was established that the measured height points on the temporary dam crown, at the bottom of the riverbed and behind the temporary dam differ by 10–20 cm in comparison with the corresponding point of the topographic layout which means that there is no sediment.

Digital 3D model based on the topographic layout in 1:1000 scale of the dam location and approximately 100 m upstream. This layout was used for planning and construction of the dam. This layout does not have data (heights) of the underwater part of the River Piva, i.e., the lowest contour line is one with height 500 m. Comparing this model with bathymetric revealed that the bottom of the River Piva former flow is located on heights 496–497 m, which means that there is no drift whose size could be correctly established, and it could be determined if bathymetric heights of the bottom bed are above 500 m. Also, comparing these two models revealed that on the left bank there is no change, while on the right one, from heights 600 m to the existing asphalt road Mratinje—Plužine, the bathymetric model gradually rises above the model of 1:1 000 to 7 m. The conclusion is that this is material that has been left during the construction of the road Mratinje—Plužine that for the most part runs through the incision and tunnels.

Digital 3D model based on topographic maps 1:25000 (TK25) with 10 m equidistance for underwater part of the reservoir, made using same methodology as bathymetric model. The mentioned map was created in the 60's of the 20th century and reflects the state of the site before the construction of the HPP Piva. Although it was known in advance that this model is less accurate than the bathymetric model and therefore not as adequate for comparison, research still took place and allowed the obtaining concrete data. Declared height accuracy of the TK25 model for flat terrain is about 3 m, in mountain areas about 5 m, and in extreme areas such as canyons and Piva Komarnica it can be up to 15 or 20 m. The main sources of TK25 errors, in this case can be classified into several categories: photogrammetric methods errors, the impact of slope, cartography generalization of the content impact, mapping technology errors, analog material deformation on which the height content was mapped and errors of digitalization of contour data

Spatial analysis and analysis of the height difference with two digital terrain models (TK25 and bathymetric model), was conducted by statistical analysis. For identical points at grid size 10×10 m,

height differences on the whole accumulation of HPP Piva were calculated on 160,000 spots. The frequency height difference histogram (Figure 7.) and statistical indicators (Table 3.) shows that the model from the topographic maps is, on average, lower than the bathymetric model for 3.13 m, and confirmed the accuracy of the declared individual elevation is ±12.66 m in 68.3% of cases. Regular spatial distribution of height difference (Gauss distribution curve) indicates high reliability of obtained data.

Furthermore, on the basis of the obtained height difference a thematic map was made that shows the height difference of these two terrain models. Figure 8. shows part of the map. It is clear that the spatial distribution of the heights differences does not indicate the objective changes, but proves that the main factor for the existence of such differences in fact the accuracy of TK25.

Based on previous analysis it can be concluded that the comparison of terrain models obtained on the basis of topographic maps and models obtained from bathymetric survey data, can reliably determine whether the whole accumulation is sedimented. On the other side local analysis can give unreliable results.

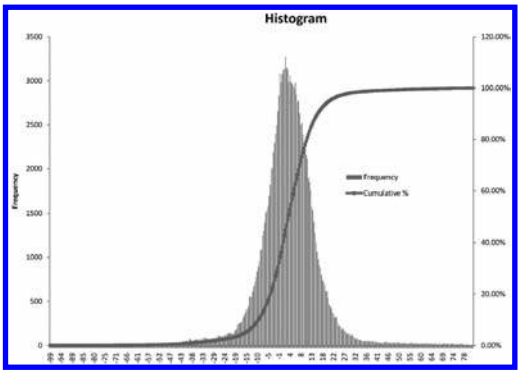

Figure 7. Height differences frequency histogram.

Table 3. Statistical indicators on height difference of two DTMs.

Mean	3.13 m
Standard error of arithmetic mean	0.03 m
Standard deviation of a single height difference	12.66 m
Sample variance	160.27 m^2
Range	180.85 m
Minimum	–98.71 m
Maximum	82.14 m
Count	150988
Confidence interval (95.0%)	0.06 m

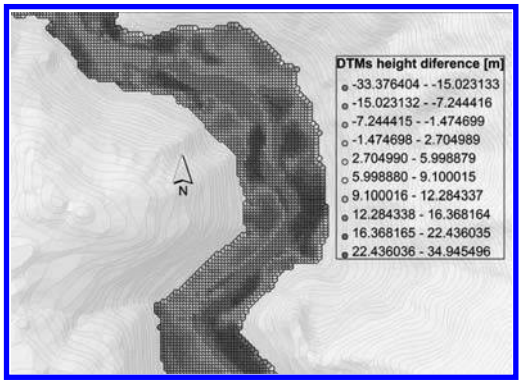

Figure 8. Spatial distribution of the height differences.

6 CONCLUSIONS AND RECOMMENDATIONS

In the basin of the Drina River in the border area of Montenegro, Serbia, Bosnia and Herzegovina there are many dams. The highest dam with the greatest accumulation is the HPP Piva. More relevant dams are Kokin Brod, Visegrad, Zvornik and Bajina Basta. Recently, dam construction has stagnated, primarily due to the limited number of technical, economic and environmentally suitable sites, and it is necessary to pay greater attention on maintaining existing ones.

One of the biggest problems in maintaining the reservoir is the control and prevention of sedimentation. Sedimentation of the reservoir's effective volume directly reduces the production capacity of the plant, and in extreme cases can lead to threats to the dam itself.

According to previous experiences and research (M. Babic-Mladenovic et al. 2003), two incomplete surveys were made on the accumulation of HE Bajina Basta in 1976. and 1989. It was established that until 2002 that the non operational volume decreased by 28.9% and the effective volume of the reservoir by 9.4%. It was determined that there is no problem related to sedimentation. Accumulation HE Kokin Brod and HE Uvac are long, relatively narrow and deep. Although it can be hardly said that they are similar to the accumulation of HPP Piva. Accumulation of Kokin Brod was built during 1962. and surveyed in 1967. and 1970. Accumulation HE Uvac was never surveyed. On the basis of scarce data it is estimated that until 2002. the reservoir lost about 3% of effective volume and that these reservoirs do not have or will not have a sedimentation problem in the near future.

Accumulation of HPP Piva was surveyed in detail with modern technology during the course of this project. The initial state of the reservoir in

terms of capacity and configuration of the relief was determined before the construction of a dam by using a map in 1:25 000 scale and was for temporary use only.

Based on the performed analysis it can be concluded that there is no sedimentation problem in HE Piva's accumulation for now. However, with detailed analysis of the morphology of underwater micro relief it was observed that at about 180 m upstream from the dam location the river bed is significantly constrained, i.e. there is high probability of underwater landslides of the right bank. There are probably more similar cases, but not so typical, and they could not easily be observed. An additional survey conducted with modern technology could obtain reliable parameters for monitoring reservoir state.

Contemporary experience and trends, in terms of protection of reservoirs, recommend periodic surveys that should be conducted on this accumulation. Therefore, the bathymetric survey of HE Piva's accumulation, which was conducted in August 2009. is considered a modern approach to reservoir protection and this measurement represents a basis for all future reliable analysis.

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Dinámica sedimentaria del extremo aguas arriba del embalse de Ribarroja (río Ebro)

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RESUMEN: Con la finalidad de estudiar la dinámica sedimentaria del embalse de Ribarroja en el río Ebro, en 2007 se realizó una campaña batimétrica de precisión mediante sonda multihaz y a partir de ella se obtuvo el modelo digital del fondo del embalse. En diciembre de 2008 se realizó una nueva batimetría de los primeros 10 kilómetros de embalse (zona de confluencia con el río Segre). Mediante la comparación de las dos batimetrías se pudo observar que un volumen de unos 170.000 m^3 de sedimento, situado inmediatamente aguas abajo de la confluencia Segre-Ebro fue desplazado 4 kilómetros aguas abajo. Cabe indicar que en el periodo comprendido entre ambas batimetrías el único episodio de avenida importante se presentó en el Ebro entre finales de mayo y principios de junio de 2008, con un caudal medio diario máximo de 1600 m^3/s, siendo superior a los 500 m^3/s durante unos 12 días. Como que el evento de avenida fue únicamente en el río Ebro, el Segre no aportó sedimento que pudiera reponer el que se había movilizado aguas abajo de la confluencia.

ABSTRACT: In order to study the sediment dynamics in the Ribarroja reservoir, at the Ebro River, a field bathymetric campaign was carried out in 2007 using a precision multi-beam probe. In December 2008 a new bathymetric campaign of the reservoir upper 10 km (zone of confluence with the Segre tributary) was undertaken. Two digital elevation models of the Ribarroja reservoir bottom were generated using the bathymetry data from the above mentioned dates. The comparison between both models showed that 170,000 m^3 of sediment, located immediately after the Segre-Ebro confluence, were conveyed 4 km downstream. It should be noted that the only major flood event in the period between bathymetry data acquisitions took place in the Ebro river, in late May and early June 2008, with a maximum daily average flow of 1600 m^3/s, exceeding 500 m^3/s flow for about 12 days. As the flood event affected only the main Ebro River, the Segre tributary contributed no sediment that could replace the one that had been moved downstream the confluence.

1 INTRODUCCIÓN

1.1 *El embalse de Ribarroja de Ebro*

El embalse de Ribarroja se encuentra en el río Ebro, en la provincia de Tarragona (Cataluña, España). Además de los caudales del río Ebro recibe la aportación de dos afluentes importantes: el Segre y el Cinca, que confluyen justo antes de entrar al embalse. Inmediatamemente aguas arriba se encuentra el embalse de Mequinenza y aguas abajo el embalse de Flix (Figura 1).

La construcción de la presa se terminó en el año 1969, su principal utilidad es la generación de electricidad aunque sus aguas también se aprovechan para la pesca, la agricultura y el abastecimiento. Además, al igual que los otros embalses, lamina las grandes avenidas que pueda sufrir el Ebro.

La presa es de gravedad y de planta recta; se sitúa aproximadamente entre las cotas 40 y 76 m sobre el nivel medio del mar. Se trata de un embalse monomíctico de morfología triangular irregular con un volumen máximo teórico de 210 hm^3. El tiempo de residencia es de unos pocos días, cosa que implica condiciones hidrodinámicas parecidas a las de un río.

Figura 1. Situación del embalse de Ribarroja en el río Ebro. Fuente: Google Maps.

1.2 *Hidrología de la zona de estudio*

De acuerdo con los datos de caudal medio anual registrados por el servicio de aforos de la Confederación Hidrográfica del Ebro, el río Ebro tiene una aportación media anual de 8009 hm^3, los ríos Cinca y Segre (unidos poco antes de la confluencia con el Ebro en la población de Mequinenza) incrementan la aportación a 14069 hm^3, en la salida del embalse de Ribarroja (periodo 48/49–91/92). El Ebro aporta en término medio el 58,82% de las entradas al embalse de Ribarroja, el Cinca el 19,3% y el Segre el 21,85% (periodo 49/50–97/98).

1.3 *Motivación del estudio*

Con la intención de seguir estudiando la dinámica sedimentaria del embalse de Ribarroja en el río Ebro, en 2007 el grupo de investigación Flumen realizó una campaña batimétrica de precisión mediante sonda multihaz y a partir de ella se obtuvo el modelo digital detallado del fondo del embalse. Un año después, en el que solo había tenido lugar un único episodio de avenida importante (ocurrido entre finales de mayo y principios de junio de 2008) se realizó una nueva batimetría de los primeros 10 kilómetros de embalse (zona de confluencia del Ebro con el río Segre).

La intención de este estudio, patrocinado por la Confederación Hidrográfica del Ebro y realizado por el grupo de investigación Flumen, fue comparar ambas batimetrías para observar la evolución del fondo del tercio superior del embalse de Ribarroja de Ebro durante el tiempo comprendido entre ambas. Con la comparación de las dos batimetrías se podría estudiar el efecto de la avenida sobre los sedimentos, su movilización y su distribución en el espacio.

2 CAMPAÑA BATIMÉTRICA 2007

La campaña batimétrica de campo se inició el 28 de octubre de 2007 y terminó a mediados de noviembre del mismo año.

2.1 *Metodología*

Los trabajos de medida del fondo del embalse se realizaron mediante dos ecosondas (multi y monohaz de doble frecuencia) instaladas en una barca con posicionamiento DGPS en tiempo real (ver Figura 2 y Figura 3). Para la toma de datos y la

Figura 2. Sensor GPS instalado en la barca de navegación.

Figura 3. Ecosonda (multihaz) instalada en la embarcación. Campaña de campo de la batimetría 2007.

planificación de las trayectorias de navegación se utilizó el software de hidrografía SEXTANT.

Debido a la utilización de las bases topográficas del ICC (Institut Cartogràfic de Catalunya), se utilizó el sistema de coordenadas UTM ED50.

2.2 *Campaña batimétrica*

Para la obtención de la batimetría del fondo del embalse se realizaron medidas por perfiles transversales y longitudinales al río a lo largo de todo su recorrido.

De acuerdo con las exigencias del trabajo (diferentes necesidades de precisión en la toma de datos), la zona de estudio se dividió en tres tramos (definidos en la Figura 4).

Zona 1: desde el puente de Mequinenza sobre el Segre, a lo largo de 10 km aguas abajo (zona de mayor interés, mayor detalle, medidas con ecosonda multihaz). Zona 2: desde el final de la zona 1 hasta 10 km aguas abajo; mediante sonda monohaz se realizaron perfiles transversales al eje del río cada 25 m, a lo largo del perfil transversal se tomaron datos cada 2 m de intervalo; también se realizaron cinco perfiles longitudinales al río. Zona 3: desde el final de la zona 2 hasta la presa de Ribarroja, se realizaron perfiles transversales al río cada 100 m, más algunos perfiles longitudinales.

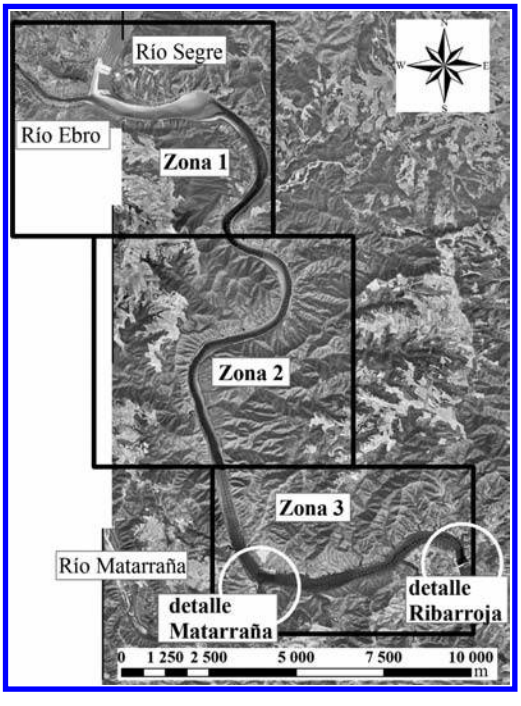

Figura 4. Subdivisión del embalse de Ribarroja por zonas de estudio: zona 1 (confluencia con el Segre), zona 2 y zona 3. Las dos circunferencias blancas muestran las dos subzonas de detalle correspondientes a la entrada del Matarraña y la zona próxima a la presa de Ribarroja.

Además de estas tres zonas se consideró necesario determinar dos subzonas de estudio más detallado dentro de la zona 3: los tramos correspondientes a la entrada del río Matarraña al Ebro y la zona más próxima a la presa. En estos tramos se densificó la toma de datos (perfiles transversales con sonda monohaz cada 10 m) para ver con detalle el comportamiento de los sedimentos en la confluencia Matarraña-Ebro y frente las tomas de la presa.

2.3 *Muestreo*

La barca siguió la trayectoria definida por las *waylines* (perfiles definidos por el usuario e introducidas previamente en el programa SEXTANT) específicas para cada tramo.

Se calibró la ecosonda, se realizaron las comprovaciones pertinentes y se realizaron los perfiles programados en la navegación.

El mismo programa SEXTANT dispone de un sistema de control de señal de satélite, y a medida que toma los datos también dispone de un filtro y control de calidad de los datos y lecturas de la sonda.

Posteriormente se procedió al almacenamiento de datos; a la transformación de los datos en formato ASCII NXYZ a archivo de AutoCAD.

2.4 *Postproceso y dibujo*

El resultado de todas las medidas realizadas a lo largo de la campaña proporcionó una nube irregular de puntos brutos (medidas originales). Posteriormente, el postproceso consistió en tratar esta nube de puntos, filtrar los errores y obtener una malla regular de puntos fácilmente tratable con programas de cálculo.

Para la zona 1 se obtuvo una malla de puntos 2×2 m, para la zona 2 se obtuvo una malla 25×2 m (después de la triangulación se generó una malla regular 2×2 m), y para la zona 3 el hecho de disponer de perfiles cada 100 m hizo imposible generar una malla regular de gran precisión por lo que inicialmente se estudiaron los perfiles individualmente. Para las dos zonas más densificadas del tercer tramo (Matarraña y presa de Ribarroja) se dispone también de una malla 2×2 m.

2.5 *Modelo digital del Embalse de Ribarroja de Ebro*

A partir de los datos proporcionados por la campaña batimétrica del fondo del embalse se generó mediante ArcGIS un modelo digital del fondo del

embalse para las zonas 1 y 2, la confluencia del Ebro con el Matarraña y las proximidades de la presa (el resultado se puede ver en la Figura 5).

Para el tratamiento de datos se utilizaron los formatos ráster y TIN en función de los requerimientos del estudio.

2.6 *Detección y corrección de artefactos. Análisis de calidad del modelo*

El modelo digital del embalse inicial presentaba una serie de artefactos y elementos extraños correspondientes mayormente a la presencia de árboles. Estos artefactos fueron corregidos de manera semiautomatizada mediante Arcmap de ESRI. El procedimiento utilizado sirvió para corregir de manera aproximada estas irregularidades en todo el modelo.

2.7 *Mejora y ampliación del modelo digital del embalse de Ribarroja*

Para poder obtener un modelo digital de todo el fondo del embalse se decidió interpolar una TIN de la zona 3, no obstante, la calidad de la zona tres sería inferior a la de las dos zonas superiores.

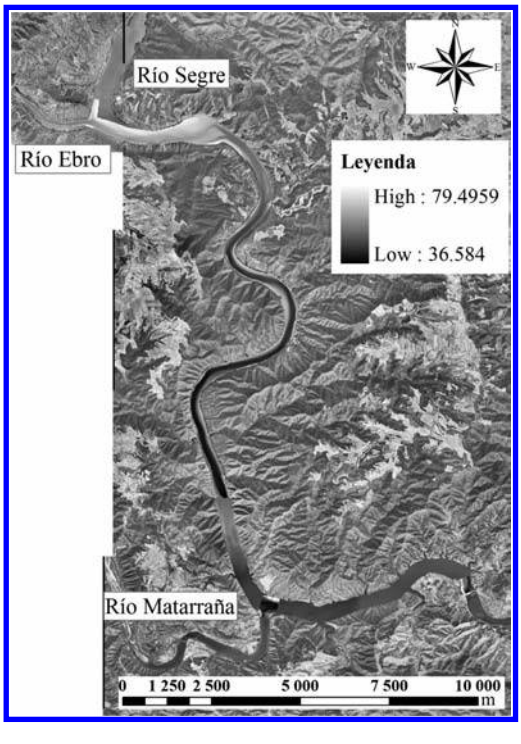

Figura 5. Primer modelo digital del terreno del embalse de Ribarroja de Ebro. Zonas 1, 2 y subzonas de la entrada del Matarraña y las cercanías de la presa de Ribarroja. Batimetría 2007.

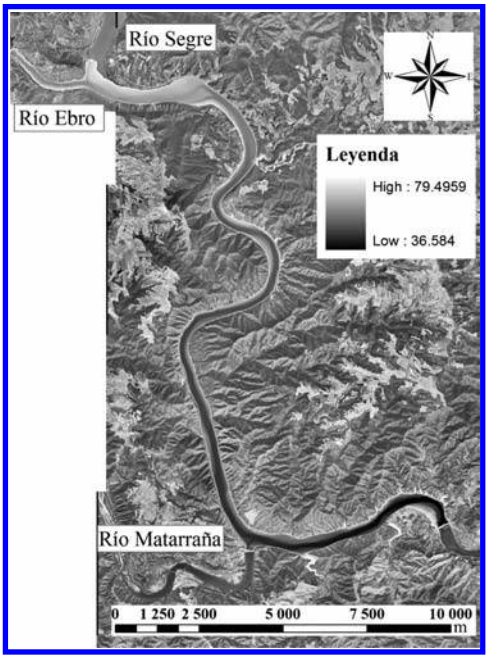

Figura 6. Modelo digital del terreno del fondo del embalse de Ribarroja de Ebro.

Además, debido a las dificultades que se presentaron en la navegación por zonas poco profundas del embalse (principalmente zonas próximas a las márgenes), no se disponía de suficiente información batimétrica en algunos laterales del río.

Para ampliar estos tramos sin información, se consideró la opción de interpolar un modelo aproximado a partir de los límites del contorno de la batimetría con las curvas de nivel disponibles de la topografía del entorno del embalse. Esta información se obtuvo en formato digital del ICC (Instituit Cartogràfic de Catalunya) y del el SITAR (Sistema de Información Territorial de Aragón).

Finalmente se obtuvo un modelo digital de embalse, de gran calidad, y para toda la batimetría debajo de la lámina de agua (el resultado se puede ver en la Figura 6).

2.8 *Análisis del modelo digital del embalse de Ribarroja*

ArcGIS permite calcular el volumen que se encuentra entre un plano horizontal a una cota determinada y el modelo digital y el área de este plano. Este volumen correspondería al volumen de agua almacenada en función de distinto s niveles de la lámina de agua. De ésta forma se calcularon las áreas y volúmenes correspondientes a distintas elevaciones para el modelo digital de la batimetría 2007, tal y como muestra la Figura 7.

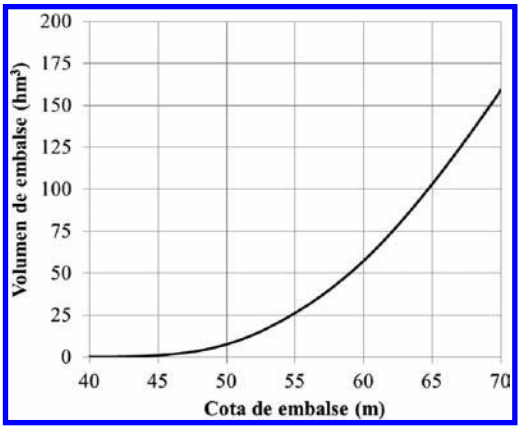

Figura 7. Modelo digital del terreno del fondo del embalse de Ribarroja de Ebro (hasta la cota 70).

De la figura citada anterior mente, el dato que resultó de especial interés fue el del volumen bajo el plano para poder ver la capacidad de almacenaje del embalse.

Cabe destacar que en este cálculo no se consideraron las zonas de embalse correspondientes a los cauces del Matarraña y del Segre.

3 CAMPAÑA BATIMÉTRICA 2008

Este trabajo se llevó a cabo análogamente a la anterior campaña batimétrica realizada en 2007. Aunque la batimetría de 2007 se realizó para todo el embalse, en 2008 solo se estudió el primer tercio superior: desde la confluencia Ebre-Segre hasta unos cinco kilómetros aguas abajo.

El trabajo se llevó a cabo entre los días 3 y 5 de diciembre de 2008, aproximadamente un año posteriormente a la anterior batimetría.

3.1 *Metodología*

En esta campaña batimétrica únicamente se utilizó la sonda multihaz. Se realizaron un perfil longitudinal (siguiendo el eje del río) y una serie de perfiles transversales al río (y consecuentemente al eje) que se pueden observar en las Figuras 8 y 10.

3.2 *Modelo digital del tercio superior del embalse*

Se disponía de un fichero con la nube de puntos XYZ de la batimetría obtenida a partir de la campaña realizada en diciembre de 2008 con ecosonda multihaz.

A partir del fichero de datos de campo se pasó a otro nuevo fichero de extensión ASCII; y a partir de éste se realizó una rasterización (con un tamaño de celda de 2×2 m) mediante el programa ArcMap de ESRI (ver Figura 8).

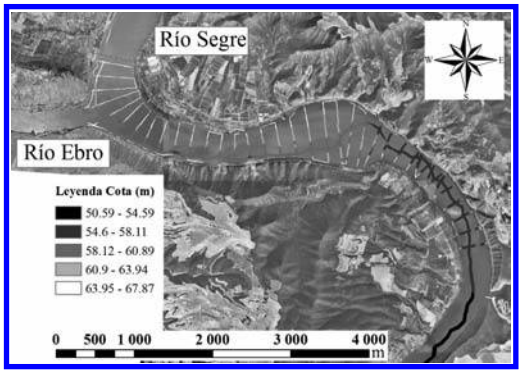

Figura 8. Rasterización de la batimetría realizada en 2008 (perfiles transversales y perfil longitudinal del eje del río).

4 AVENIDA 2008

En el periodo comprendido entre la realización de las batimetrías 2007 (octubre–noviembre de 2007) y 2008 (diciembre de 2008), aproximadamente un año, solo tuvo lugar un único episodio de avenida importante.

La avenida se presentó principalmente en el río Ebro entre finales de mayo y principios de junio de 2008 (Figura 9), con un caudal medio diario máximo a la salida del embalse de Ribarroja de 1600 m³/s, siendo superior a los 500 m³/s durante unos 12 días (datos cedidos por ENDESA). Como que el evento de avenida fue únicamente en el río Ebro, el Segre no aportó sedimento que pudiera reponer el que se había movilizado aguas abajo de la confluencia.

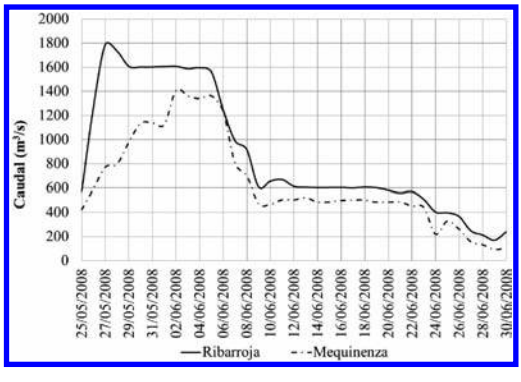

Figura 9. Caudales medios diarios desaguados por las presas de Mequinenza (inmediatamente aguas arriba del embalse de Ribarroja de Ebro y de la confluencia Ebre-Segre) y los desaguados por la presa de Ribarroja. En línea contínua se pueden ver los caudales medios diarios de salida por la presa de Ribarroja, en línea discontínua los caudales medios diarios desaguados por la presa de Mequinenza. Datos cedidos por ENDESA.

5 ESTUDIO DE LA VARIACIÓN BATIMÉTRICA ENTRE 2007 Y 2008

Este apartado tiene por objeto analizar la dinámica de los sedimentos depositados en el extremo aguas arriba del embalse de Ribarroja:

- Comparación de la batimetría Batimetría 2008 con la Batimetría 2007: Inicialmente se realizó una comparación directa mediante la superposición de los dos modelos digitales (estudio por perfiles) y posteriormente mediante la resta de las dos batimetrías (modelo digital de erosión/sedimentación).
- Evaluación del sedimento acumulado y/o erosionado en el periodo comprendido entre ambas batimetrías. Además de estimar las cantidades movilizadas también se compara su distribución espacial.

5.1 *Comparación de batimetrías*

Para analizar las dos batimetrías realizadas en 2007 y 2008, se procedió inicialmente a comparar directamente los datos de ambas batimetrías. Esto únicamente se podía hacer para las zonas que tenían información batimétrica de la campaña 2008 (basada en un perfil longitudinal del eje del río y una serie de perfiles transversales a éste).

Mediante la extensión Easy Profiler de ArcGis se obtuvieron una serie de perfiles transversales y otro longitudinal (correspondiente al eje profundo del río) para cada una de las dos batimetrías. La localización y definición de estos perfiles venía definida por la propia batimetría 2008 como se puede observar en la Figura 10.

En los primeros kilómetros aguas arriba se observa una clara tendencia erosiva y posteriormente una tendencia más deposicional (aproximadamente a partir del perfil 25).

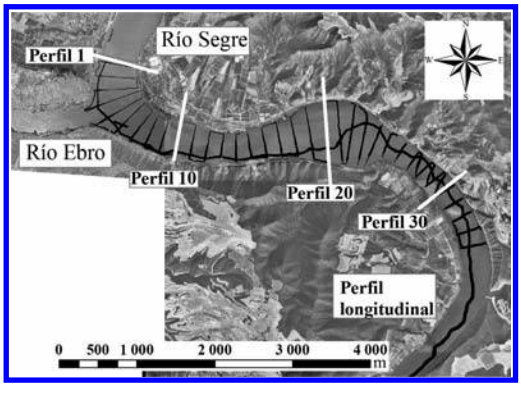

Figura 10. Definición de los perfiles longitudinal y transversales utilizados para el estudio de las batimetrías 2007 y 2008.

Las diferencias observadas son de pocas decenas de centímetros (de unos 30 cm como máximo). Además en los perfiles transversales se puede ver que estas tendencias son más acusadas en la zona del perfil correspondiente al eje profundo más que en los perfiles laterales. Esto se debe a que por el eje profundo hay más velocidad y circulación de los caudales más importantes

Como ejemplos, se muestran un perfil de la zona superior y otro de la zona erosiva inferior: en el perfil número 9 (Figura 11) se puede ver como la Batimetría 2007 queda por encima de la 2008, lo que indica una tendencia erosiva; en cambio, más aguas abajo (a unos 5 kilómetros de la confluencia Segre-Ebro) se puede ver como lo que sucede es justamente lo contrario en el perfil 31 (Figura 12).

5.2 *Diferencia de batimetrías*

Para la primera comparación entre ambas batimetrías se procedió a restar la batimetría original (2007) de la batimetría final (2008) para poder

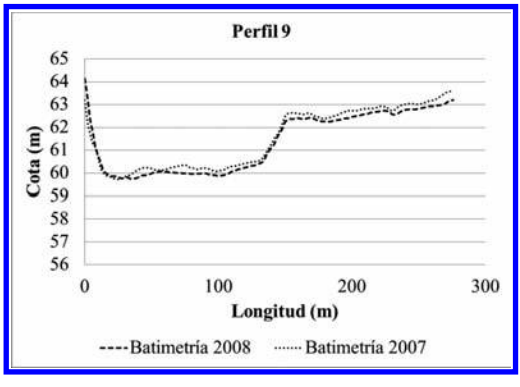

Figura 11. Perfil 9, comparación entre las dos batimetrías.

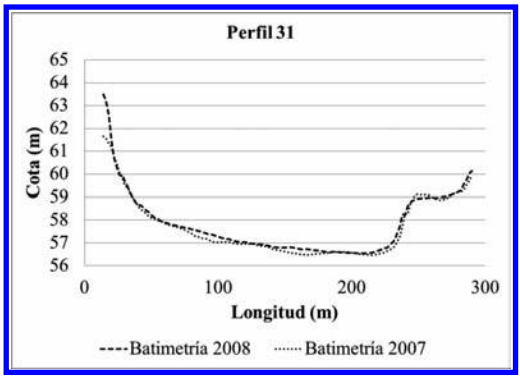

Figura 12. Perfil 31, comparación entre las dos batimetrías.

ver claramente las zonas de erosión y las zonas de deposición, además así se podrían cuantificar también las magnitudes de las diferencias.

La diferencia de batimetrías se realizó mediante el programa ArcMap, restando el ráster de la batimetría 2007 del de la batimetría 2008, el resultado se puede observar en la Figura 13. En la Figura 14 se definen la zona erosiva (en negro) y la zona deposicional (en blanco) superpuesta en la ortofotografía del tramo de estudio.

Por lo correspondiente al eje profundo, a continuación, en la Figura 15, se muestra la diferencia de los dos ejes profundos correspondientes a la Batimetría 2008 y a la Batimetría 2007.

5.3 *Variación espacial del sedimento erosionado o acumulado*

Para observar la distribución espacial del sedimento, se realizó un estudio dividiendo el embalse en segmentos de longitud constante, en los que se ha calculó la altura media sedimentada o erosionada; posteriormente se multiplicó esta media por la superficie del subtramo y así se obtuvo el volumen medio erosionado (o sedimentado) para cada uno de los subtramos. Con tal de que dentro de cada uno de los subtramos hubiera suficiente información batimétrica se consideraron segmentos de 200 metros de longitud perpendicularmente al eje.

En la Figura 16 se muestra el volumen sedimentado o erosionado en los diferentes subtramos,

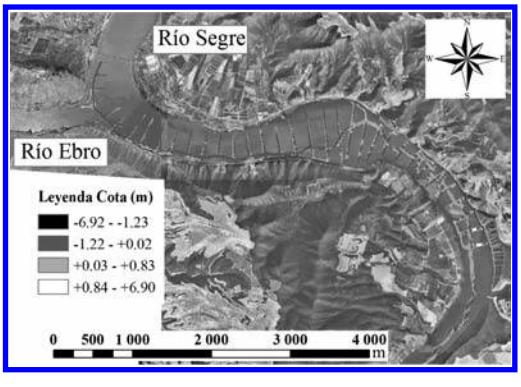

Figura 13. Diferencia (en metros) de la batimetría 2007 menos la 2008. A colores más oscuros mayor ersosión, colores más claros mayor deposición.

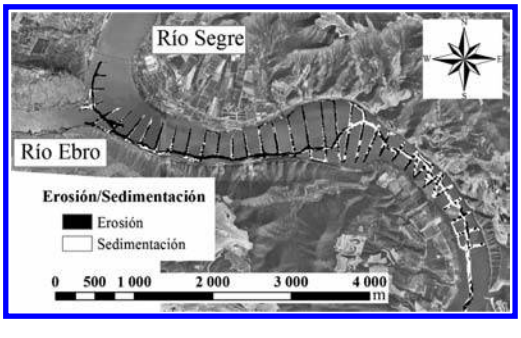

Figura 14. Zonas de erosión (negro) y zonas de deposición (blanco). Se puede observar claramente que los primeros 4000 metros aguas abajo de la confluencia Segre-Ebro son mayormente erosivos, mientras que aguas abajo la tendencia es deposicional.

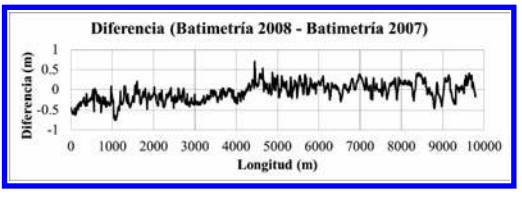

Figura 15. En este gráfico se puede observar la diferencia de los dos modelos digitales del fondo de embalse (batimetría 2008—batimetría 2007). Se puede ver que para los primeros 4000 m los valores son negativos, por lo que indican erosión; a partir de aquí, la diferencia es positiva, indicando zonas deposicionales. Diferencia en metros.

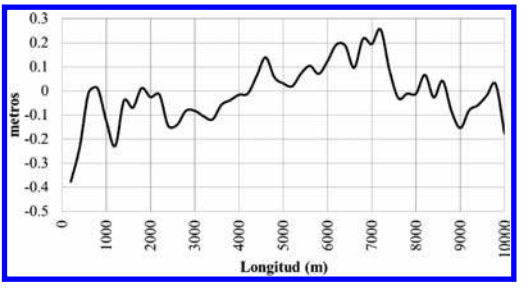

Figura 16. Acumulación media de sedimento por subtramo; las zonas de erosión (negativas) y zonas de deposición (positivas). Valores en metros.

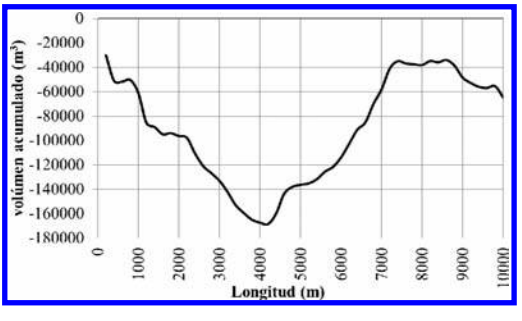

Figura 17. Curva de sedimento acumulado (m^3). Se puede observar que en los primeros 4000 m se produce un volumen de erosión igual al que se acumula en los 4000 m aguas abajo. Esto indica que el sedimento se desplazó unos 4000 m hacia aguas abajo.

mientras que en la Figura 17 se analiza la variación acumulada del volumen de sedimento.

Un aspecto de interés que puede observarse en la Figura 17 es que en los primeros 4000 m se produce un volumen de erosión relativamente similar al que se sedimenta en los 3000 m siguientes, o dicho de otro modo: en esta zona del embalse un volumen de unos 170.000 m^3 de sedimento ha sido desplazado unos 4 km aguas abajo. Todo ello está directamente relacionado con el avance del frente de la masa de sedimento situada en los primeros 4000–5000 m del embalse (ver Figuras 10 y 17).

6 CONLUSIONES

A continuación se enumeran y resumen los principales aspectos a destacar del estudio de las batimetrías digitales del terreno realizados en 2007 y 2008 en el embalse de Ribarroja de Ebro.

Respecto a la dinámica de los sedimentos en el extremo aguas arriba del embalse de Ribarroja, se compararon las batimetrías realizadas en 2007 (para todo el embalse de Ribarroja de Ebro) y la de 2008 (estudiando el comportamiento del perfil del eje del río y de los perfiles transversales para el tercio superior del embalse). Mediante la comparación de las dos batimetrías (ver apartado 5) se ha podido evaluar la cantidad y la disposición espacial del sedimento acumulado o erosionado a lo largo del periodo.

En la Figura 16 se muestra la variación acumulada del volumen de sedimento a lo largo del tramo. Puede observarse que en los primeros 4000 m se produce un volumen de erosión relativamente similar al que se sedimenta en los 3000 m siguientes, o dicho de otro modo: en esta zona del embalse un volumen de unos 170.000 m^3 de sedimento ha sido desplazado unos 4 km aguas abajo.

Todo ello está directamente relacionado con el avance del frente de la masa de sedimento procedente del Segre que ha ido llegando al embalse de Ribarroja y que se queda justo aguas abajo de la conflulencia de los dos ríos. Por este motivo la lengua de sedimento se sitúa en los primeros 4000–5000 m del embalse (ver Figura 7), y ésta solo se removiliza en episodios de avenida importantes, cuando se supera cierto umbral de caudales y el agua tiene fuerza tangencial suficiente para resuspender el sedimento.

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Caracterización física y química del sedimento superficial del embalse de Ribarroja

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RESUMEN: Para estudiar la heterogeneidad espacial de la estructura física y química del sedimento superficial del embalse de Ribarroja se ha analizado la granulometría y la concentración de elementos mayoritarios (Al, Fe, Si, K, Ca, Mg, Ti, Mn, P) y traza (Mo, Nb, Zr, Y, Sr, Rb, Th, Pb, Sn, Ce, Ga, Zn, W, Cu, Co, Ni y V) en dieciséis puntos de muestreo distribuidos a lo largo del embalse. En conjunto, el sedimento es bastante homogéneo: la textura es limo-arcillosa y la composición mayoritaria corresponde a una mezcla de carbonatos y silicatos meteorizados, con moderados niveles de nitrógeno y fósforo. Las concentraciones de metales traza son moderadas-bajas, no superando el nivel de toxicidad probable, aunque níquel y cromo sobrepasan el nivel umbral. El análisis de componentes principales, realizado sobre la concentración normalizada de metales, indica como factores principales de variación el grado de alteración-meteorización de los materiales sedimentados y la asociación de ciertos metales a los aglomerados de arcillas y materia orgánica.

ABSTRACT: To determine the spatial heterogeneity of physical and chemical characteristics in superficial sediments from ribarroja reservoir, we have analyzed particle size distribution, and concentrations of major (Al, Fe, Si, K, Ca, Mg, Ti, Mn, P) and minor (Mo, Nb, Zr, y, Sr, Rb, Th, Pb, Sn, Ce, Ga, Zn, W, Cu, Co, Ni y V) elements in sixteen sites distributed along reservoir. globally, the sediment was quite homogeneous: grain size was dominated by silt and clay, and major composition agreed with a mixture of carbonates and weathered silicates, with moderate levels of nitrogen and phosphorus. Concentrations of minor elements did not surpass the Probable Effect Levels, although nickel and chromium exceeded the Threshold Effect Level. Principal Component Analysis performed on normalized minor elements indicated as main processes determining the variation of sediment composition the degree of alteration/weathering of materials and the association of some metals to the organic/clay aggregates.

1 INTRODUCCIÓN

Los embalses desempeñan un papel crucial en el transporte y sedimentación de partículas a lo largo del cauce fluvial, como consecuencia de la alteración en el régimen hidrológico y de la retención de materiales favorecida por la presencia de presas. La retención de la carga en suspensión puede superar en algunos casos el 95%, lo que puede suponer consecuencias negativas como la disminución de la capacidad de almacenamiento hídrico y el cambio en la naturaleza y cantidad de los sedimentos aportados al tramo fluvial posterior a la presa (Loizeau et al., 1997). Por otra parte, dependiendo de las características biogeoquímicas del embalse, la acumulación de sedimentos puede implicar también consecuencias favorables para la utilización de los recursos hídricos: así, la retención de fósforo sedimentario contribuye a reducir la eutrofización en el tramo fluvial posterior a la presa (López et al., 2009).

Aunque el material acumulado en el sedimento corresponde a la sedimentación de la carga en suspensión en el agua, las características físicas y químicas de ambos materiales son distintas. Los procesos de selección durante el transporte, debidos a la deposición selectiva de las partículas según su tamaño y energía de transporte, conllevan una considerable heterogeneidad espacial en la estructura física del sedimento. La presencia de presas afecta principalmente al transporte de partículas superiores a 4 micras, mientras

que el transporte de arcillas (partículas menores de 4 micras) se ve relativamente poco afectado (Anselmetti et al., 2007). Además, en el sedimento se producen numerosos procesos biogeoquímicos que modifican la naturaleza y estructura del material sedimentado (López, 2009), de forma que cuanto mayor es el número de procesos implicados, mayor es la heterogeneidad en la composición química del sedimento (Kienel & Kumke, 2002; Kumke et al., 2005).

En muchos casos, en el sedimento se pueden acumular contaminantes y nutrientes en concentraciones superiores a las observadas en la columna de agua. Dada la persistencia de dichos materiales en el compartimento sedimentario, su monitorización resulta especialmente útil para evaluar la calidad ecológica de los sistemas acuáticos (Swartz et al., 1982). Además, puesto que el equilibrio entre formas disueltas y particuladas depende del entorno fisico-químico, el sedimento puede devenir una fuente de materiales indeseados hacia el agua, cuando las condiciones fisico-químicas se ven alteradas ya sea por cambios hidrodinámicos, biológicos o antropogénicos (Abrams & Jarrell, 1995; Nurnberg, 1999). Ello añade aún mayor interés al estudio de las características del sedimento como herramienta indispensable para disponer de la información necesaria a la hora de gestionar eficientemente los sistemas acuáticos.

El embalse de Ribarroja, ubicado en el río Ebro, recibe material en suspensión procedente de tres entradas principales: la salida del embalse de Mequinenza (también en el curso del Ebro), el río Segre y el río Matarraña. Dado que la presa de Mequinenza retiene mas del 95% del material en suspensión transportado por el Ebro, el aporte principal al sedimento de Ribarroja es presumiblemente el de origen autóctono (formado en el propio embalse) y el procedente de los ríos Segre y Matarraña, lo que podría determinar una heterogeneidad espacial importante. El presente estudio tiene como objetivo principal proporcionar una descripción detallada de la variabilidad espacial de la estructura física y las principales características químicas del sedimento superficial del embalse de Ribarroja. La información obtenida ha de servir como base de referencia para evaluar la calidad presente y futura del sedimento de este embalse y contribuir a la decisión de la adecuada estrategia de gestión a seguir.

2 MATERIAL Y MÉTODOS

La toma de muestras de sedimento se realizó en abril del 2008. En total se seleccionaron 16 puntos de muestreo (Tabla 1) en base a los resultados preliminares del estudio de la batimetría de precisión del embalse. Dado que los resultados disponibles indicaban la existencia de una marcada lengua de sedimento procedente del río Segre, dicha zona se muestreó de forma más exhaustiva a fin de caracterizarla con precisión. La posición de la toma de muestras se fijó mediante GPS. El sedimento superficial (0–4 cm) se recogió mediante una draga tipo Lenz de 400 cm^2 de área. Además, en siete puntos en los que se extrajo suficiente profundidad de sedimento, las muestras se subdividieron "in situ", cada 4 cm de profundidad a fin de obtener una estima de la variabilidad en profundidad. Las muestras obtenidas fueron repartidas en tres alícuotas, guardadas en frascos y/o bolsas de PVC y conservadas en frío hasta su análisis en el laboratorio.

En el laboratorio, dos alícuotas se destinaron al análisis granulométrico que se realizó con un analizador de partículas Beckman—Couter LS 230. Una alícuota se analizó directamente tras dispersión con pirofosfato y ultrasónidos, y la otra fue sometida

Tabla 1. Descripción y localización de los puntos de muestreo.

Estación	Descripción	Latitud	Longitud	Z metros
A	Cola del embalse	273791	4582544	10,9
B	Río Segre	274711	4582674	3,5
C	Frente de sedimento	275271	4582104	8,9
D	Frente de sedimento	275261	4582174	7,6
E	Frente de sedimento	276881	4582034	9,7
F	Frente de sedimento	276911	4582404	5,4
G	Frente de sedimento	277571	4582504	8,2
H	Tramo medio	278681	4581554	12,6
I	Tramo medio	279181	4580514	15,3
J	Tramo medio	279881	4577634	17,4
K	Tramo medio	277311	4573984	18,2
L	Tramo medio	278331	4570244	21,3
M	Entrada Matarraña	279351	4568714	17,5
N	Tramo presa	281101	4568614	24,2
O	Tramo presa	283211	4569654	26,3
P	Tramo presa	283921	4569894	26,3

a la eliminación de materia orgánica por adición de H_2O_2 y posteriormente se procesó igual que la submuestra sin tratar. La otra alícuota fue secada hasta peso constante para la determinación de peso seco. Posteriormente, el material seco fue molturado mediante un mortero de ágata para su completa homogeneización. La concentración de C y N se determinó sobre el material seco mediante un autoanalizador elemental CN. Para la determinación de los elementos mayoritarios se utilizó Fluorescencia de Rayos X (FRX) tras fusión a 1100°C con tetraborato de Litio. La concentración de metales traza se determinó también por FRX tras aglutinación con Elvactite 2044 y posterior prensado.

3 ESTRUCTURA FÍSICA DEL SEDIMENTO

En la mayor parte del embalse, los sedimentos son de textura limo-arcillosa, con un porcentaje de limos (4 μm < ϕ < 63 μm) que varia entre el 56% y el 74% y un porcentaje de arcillas (ϕ < 4 μm) que oscila entre el 18% y el 43%. Excepcionalmente, las estaciones G, J y D presentan un porcentaje relativamente elevado de arenas (≈10%) en detrimento de la proporción de limos (Fig. 1).

En relación a la textura del sedimento cabe destacar la notable similitud entre los sedimentos ubicados en el tramo próximo a la presa (estaciones L a P) y la estación ubicada en la cola del embalse (estación A). En el tramo de meandros (estaciones H a K) el sedimento muestra una textura similar, pero con un porcentaje de limos ligeramente superior. En el tramo superior, desde la entrada del río Segre hasta el inicio de la zona meandriforme (estaciones C a K), aparece la mayor variabilidad de la textura sedimentaria, con estaciones de tipo limo-arcilloso (estación C) y otras de tipo arcillo-arenoso como la estación G, que puede identificarse con la zona más frontal de la entrada de sedimento procedente del río Segre.

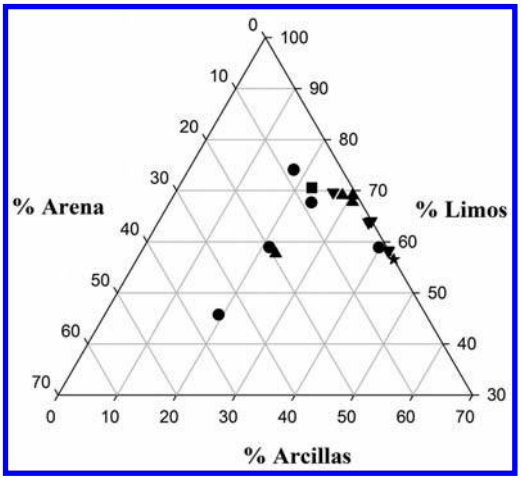

Figura 1. Diagrama de Shepard que indica la composición porcentual de arcillas, limos y arenas en los sedimentos superficiales de Ribarroja. Los símbolos indican los distintos tramos. Estrella: estación A; Cuadrado: estación B; Círculos: estaciones C a G; Triángulos: estaciones H a K; Triángulos invertidos: estaciones L a P.

Los parámetros estadísticos de la distribución del tamaño de partículas observada en cada muestra dependen de factores como el tiempo de transporte o el tipo y la energía del ambiente de deposición (Stevens et al., 1996; García et al., 2005). Los elevados valores de la desviación estándar obtenidos (Tabla 2) son indicativos de un ambiente sedimentario altamente heterogéneo, en especial en aquellas estaciones en las que aparecen los máximos valores (D, G y J).

En todo el embalse, la distribución de tamaños de partículas presenta asimetría positiva, indicando que el sedimento se encuentra formado por una mezcla de materiales con dominancia de los materiales finos. Además, en Ribarroja la asimetría aparece estrechamente relacionada con la curtosis que, al igual que la desviación estándar, da una idea del grado de selección del material sedimentado. La estación A es la que presenta los menores valores de asimetría y curtosis (Tabla 2), lo que indica que se trata de una zona con baja selección del material que determina una mezcla de partículas no dominada por una fracción concreta. Por el contrario, las estaciones inmediatamente posteriores a la entrada del río Segre (estaciones C y E) y a la entrada del Matarraña (estación N) son las que presentan mayores valores de estos parámetros

Tabla 2. Parámetros estadísticos de la distribución de tamaños de partícula en el sedimento superficial.

Estación	Media micras	SD	Asimetría	Curtosis
A	7,2	7,5	2,1	5,8
B	20,0	26,9	2,8	9,4
C	9,5	14,5	4,6	28,1
D	46,3	92,1	3,6	14,0
E	22,9	42,1	4,5	25,3
F	23,1	28,0	2,5	7,3
G	106,9	187,9	2,8	8,9
H	13,3	17,9	3,7	18,5
I	11,2	14,1	3,7	20,4
J	43,9	83,0	3,6	14,7
K	10,4	11,5	3,5	21,0
L	8,1	8,5	2,5	8,9
M	7,8	9,1	2,7	9,7
N	9,9	11,3	3,8	24,4
O	15,3	20,3	3,1	12,1
P	7,5	8,5	2,7	9,1

indicando que se trata de zonas con una sedimentación altamente selectiva de materiales muy finos.

La distribución del tamaño de partículas en el sedimento está relacionada con el contenido de materia orgánica del mismo, ya que la materia orgánica puede incorporarse en los microagregados de arcillas y limos, o bien adsorberse sobre su superficie (Zonta et al., 1994). La comparación entre el espectro de tamaños en una muestra de sedimento sin tratar y en otra de la misma ubicación a la que se ha eliminado la materia orgánica, permite identificar a que tamaño de partículas se encuentra unida preferentemente la materia orgánica (Fig. 2). Los resultados obtenidos indican que una parte de los limos se encuentran formados por la asociación de partículas inorgánicas de pequeño tamaño (fracción arcillosa) cohesionadas por la interacción con la materia orgánica. Cuando ésta es eliminada, la fracción de limos disminuye porcentualmente al tiempo que aumenta la fracción arcillosa de menor tamaño (Fig. 2). Este patrón aparece en la mayor parte del embalse, pero es especialmente evidente en las estaciones situadas en el tramo final (estaciones K a P). Sin embargo, en algunas estaciones ubicadas en el tramo superior y medio (D, E, G y J) la disminución en la proporción de limos al eliminar la materia orgánica no va asociada al aumento de la fracción más fina, sino a un aumento de la fracción gruesa. Aunque estos resultados deben interpretarse con precaución por tratarse de estaciones que se caracterizan por su elevada heterogeneidad, el cambio de composición porcentual observado podría corresponder a la presencia de abundantes partículas orgánicas, de tamaño similar al de los limos y arcillas (es decir de tamaño inferior a 63 µm) mezcladas con arenas de naturaleza inorgánica. Ello sugiere pues la acumulación de materia orgánica de pequeño tamaño en los ambientes sedimentarios de mayor heterogeneidad del tramo superior del embalse.

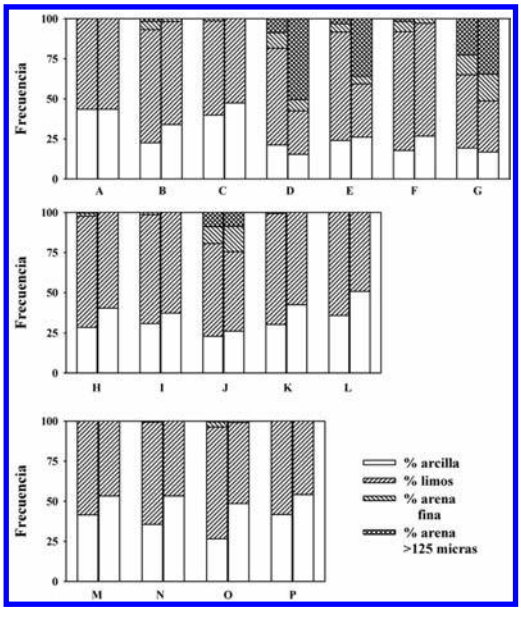

Figura 2. Porcentaje de arcillas, limos y arenas en réplicas sin tratar (izquierda) y réplicas a las que se ha eliminado la materia orgánica (derecha).

4 CARACTERIZACIÓN QUÍMICA DEL SEDIMENTO SUPERFICIAL

4.1 *Elementos mayoritarios*

La composición mayoritaria del sedimento del embalse de Ribarroja se corresponde con la de una mezcla de silicatos meteorizados y carbonatos. Silicio, aluminio y potasio, los principales componentes de los silicatos meteorizados, se encuentran en una concentración elevada, mientras que la concentración de calcio, elemento asociado principalmente a los carbonatos, es del mismo orden que la de silicio (Tabla 3).

Por lo que se refiere a los elementos asociados a la materia orgánica, las concentraciones de nitrógeno y fósforo presentan valores propios de sistemas moderadamente eutróficos (López & Morguí, 1993), con una relación estequiométrica N/P que oscila entre 3 y 5 (expresada en átomos), lo que indica que el sedimento se halla empobrecido en N con respecto a la materia orgánica. Cabe destacar que la relación P/Al en el sedimento superficial presenta un valor medio de 16. Tratándose de sedimentos con presencia importante de carbonatos, este valor sugiere que la capacidad de retención de

Tabla 3. Concentración media, desviación estándar y coeficiente de variación (CV = Media*100/SD) de los elementos mayoritarios analizados en el sedimento superficial de Ribarroja.

		Media	SD	CV
C_{total}	%	5,5	0,6	10,7
N_{total}	%	0,2	0,1	22,0
P	$mg \cdot g^{-1}$	1,2	0,2	15,6
Si	$mg \cdot g^{-1}$	171,4	15,4	9,0
Ca	$mg \cdot g^{-1}$	138,0	13,4	9,7
Al	$mg \cdot g^{-1}$	69,1	11,8	17,0
Fe	$mg \cdot g^{-1}$	32,8	4,7	14,4
Ti	$mg \cdot g^{-1}$	3,0	0,3	11,0
K	$mg \cdot g^{-1}$	21,0	3,0	14,5
Mg	$mg \cdot g^{-1}$	12,9	1,5	11,8
Mn	$mg \cdot g^{-1}$	0,6	0,1	11,3

fósforo por el sedimento se halla muy cercana a la saturación (Jensen et al., 1992; López et al., 2006).

El análisis de submuestras tomadas de distintas profundidades (entre 0 y 10 cm) muestra que el sedimento superficial es prácticamente homogéneo en lo relativo a los elementos mayoritarios, siendo el coeficiente de variación inferior al 3% para todos los elementos analizados. Con respecto a la variación longitudinal observada a lo largo del embalse, aluminio, potasio y hierro muestran una variación muy similar presentando concentraciones uniformes y elevadas en el tramo inferior (estaciones K a P). En la cabecera y en el río Segre (estaciones A a C) se observan concentraciones parecidas pero más varia bles, mientras que el tramo intermedio (C a K) es el que presenta una mayor variabilidad, con concentraciones inferiores a las del resto del embalse (Fig. 3A). Calcio y silicio siguen un patrón de variación algo distinto, observándose las mayores concentraciones en la zona comprendida entre las estaciones D y K, con máximos de silicio muy acusados en las estaciones D y G (Fig. 3B). Nitrógeno y fósforo tienden a acumularse en el tramo inferior, donde se alcanzan valores próximos a 3 $mg \cdot g^{-1}$ de N y 1,5 $mg \cdot g^{-1}$ de P (Fig. 3C). En cuanto a la concentración de carbono varía de forma muy similar a la de calcio, observándose las mayores concentraciones, 6,80%, en las estaciones F y G. Ello sugiere que la variabilidad de carbono en el sedimento de Ribarroja está más asociada a la fracción inorgánica que a la fracción orgánica. Aún así, el cociente elemental entre carbono y calcio es siempre superior a 1, indicando el exceso sobre 1 la fracción de carbono asociada a la materia orgánica. Este valor es en promedio superior en el tramo final (estaciones K a P), aunque presenta también dos máximos significativos en la estación B (río Segre en la confluencia con el embalse) y en la estación G (Fig. 4A).

El análisis de los cocientes elementales proporciona información adicional acerca de la dinámica biogeoquímica del sedimento (López et al., 2006). Así, el cociente Si/Al es una estimador indirecto de la proporción entre cuarzo, asociado a la sedimentación de materiales más gruesos y que requieren una mayor energía de transporte, y arcillas asociadas a la sedimentación de materiales mas finos que requieren poca energía para ser transportadas. La variabilidad del cociente Si/Al a lo largo del embalse (Fig. 4B), indica que la mayor proporción de cuarzo aparece en las estaciones intermedias, en especial en las estaciones G, D y J, que serían las zonas en que sedimentan los materiales de mayor tamaño. Ello concuerda con la distribución

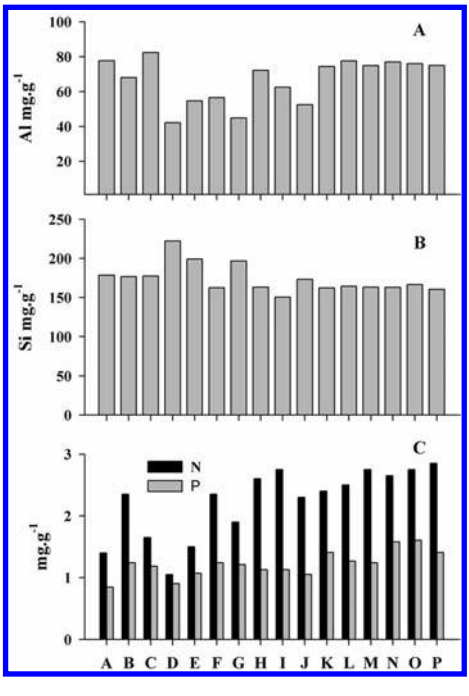

Figura 3. Variación espacial a lo largo del eje del embalse de la concentración de aluminio (A), silicio (B) y nitrógeno y fósforo (C) en el sedimento superficial.

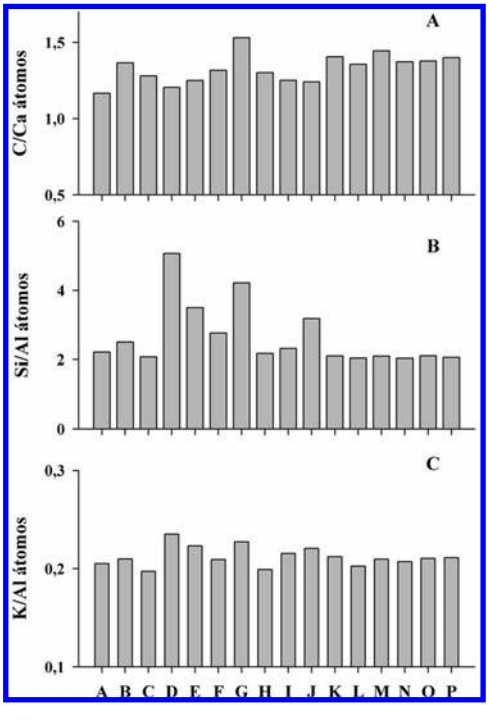

Figura 4. Variación espacial a lo largo del eje del embalse de los cocientes elementales C/Ca (A), Si/Al (B) y K/Al (C) en el sedimento superficial.

granulométrica comentada en el capítulo precedente. El cociente K/Al está muy relacionado con la naturaleza y grado de meteorización de las arcillas. En las rocas el valor medio se encuentra alrededor de 0,24, mientras que en el material altamente meteorizado, habitualmente transportado por los ríos, está relativamente empobrecido en K al perderse este elemento por movilización (K/Al = 0,15) (Viers et al., 2009). El sedimento de Ribarroja presenta un valor intermedio, 0,21, más próximo al contenido de las rocas, indicando la presencia de arcillas con un grado intermedio de meteorización. Destaca el hecho de que los valores más elevados se encuentran en el tramo de meandros (estaciones D a K), donde se encontrarían los materiales menos alterados, mientras que en el tramo final y en la cabecera del embalse se observan cocientes más bajos indicando la presencia de arcillas más alteradas (Fig. 4C).

4.2 *Elementos traza*

En la Tabla 4 se dan las concentraciones medias de los elementos traza analizados en el sedimento superficial de Ribarroja. Entre los elementos analizados, los que se encuentran asociados con mayor frecuencia a la contaminación antropogénica agrícola y/o industrial son zinc (Zn), cobre (Cu), níquel (Ni), plomo (Pb) y cromo (Cr), que pueden presentar efectos adversos sobre la biota cuando su concentración excede determinado límite. Como puede apreciarse, las concentraciones observadas son inferiores a las descritas para el material transportado por los ríos, mientras que el factor de enriquecimiento con respecto a la roca caliza (cociente entre la concentración en sedimento y la concentración en la roca) es mayor para Cu, Ni y Cr (7,5, 6 y 5,8 respectivamente) que para Zn y Pb (3,5 y 1,8 respectivamente). Al comparar los resultados obtenidos con los valores críticos de umbral de toxicidad (TEL) y nivel de toxicidad probable (PEL) utilizados por otros autores (Evans and Gottgens, 2007) cabe destacar que Zn y Pb se encuentran siempre por debajo del nivel considerado como umbral de toxicidad (123 $mg \cdot g^{-1}$ y 35 $mg \cdot g^{-1}$, respectivamente); los niveles de Cu se aproximan al umbral de toxicidad (35.7 $mg \cdot g^{-1}$) en las estaciones L y M (entrada del Matarraña y estación inmediatamente anterior) y son inferiores en el resto del embalse, mientras que las concentraciones de Cr y Ni son superiores al nivel umbral de toxicidad (37 $mg \cdot g^{-1}$ y 18 $mg \cdot g^{-1}$ respectivamente) en todo el embalse, aunque en ningún punto llega alcanzarse el nivel de toxicidad probable (90 $mg \cdot g^{-1}$ y 36 $mg \cdot g^{-1}$ respectivamente).

Tabla 4. Concentración media de elementos traza en el sedimento superficial de Ribarroja. Datos en $\mu g.g^{-1}$ Se dan también como comparación las concentraciones de estos elementos en distintos tipos de materiales: 2) Concentración media en el sedimento en suspensión de ríos del mundo (Viers et al., 2009). 3) Concentración media en rocas calcáreas (a: tomado de Guo et al., 1998; b: tomado de Potts et al., 1992).

	Media	2	3	
Zn	105,5	208,0	30	b
Cu	30,2	75,9	4	b
Ni	30,9	74,5	5	b
Pb	24,0	61,1	13	b
Cr	56,8	130,0	10	b
Mo	2,2	3,0	0,5	a
Nb	13,0	13,5	4,1	a
Zr	97,5	160,0	29	a
Y	17,3	21,9	3,9	a
Sr	457,8	187,0	218,0	a
Rb	114,4	78,5	21	a
Th	11,9	12,1	1,6	a
Ga	14,9	18,1	2,8	a
W	3,7	2,0	0,3	a
V	97,6	129,0	14	a
Ce	40,1	73,6	–	–
Co	14,8	22,5	2,1	a
Ba	318,6	522,0	374	a

Para integrar toda la información disponible acerca de la concentración de elementos traza en el sedimento de Ribarroja, se ha realizado un análisis de componentes principales (PCA), a fin de reducir el elevado numero de variables a unas pocas variables combinadas, que integran a las anteriores y que pueden asociarse con los principales procesos que determinan la composición química del sedimento. Siguiendo a Kumke et al. (2005) el PCA se ha efectuado sobre las concentraciones normalizadas con respecto al aluminio.

El análisis efectuado ha permitido extraer cuatro componentes principales que explican un 84% de la varianza. La primera componente explica el 40% de la varianza y aparece determinada principalmente por la concentración de niobio (Nb), zirconio (Zr), ytrio (Y), plomo (Pb) y zinc (Zn) que presentan factores de carga superiores a 0,85, y en menor grado por vanadio (V) y cobalto (Co) (factor de carga > 0,75). Esta componente se encuentra además estrechamente correlacionada con el cociente elemental Si/Al ($r^2 = 0.870$, $p < 0,005$).

Todos los elementos asociados a la componente 1 se caracterizan por presentar concentraciones relativas al aluminio elevadas en las calizas dolomíticas y en la corteza terrestre, mientras que las arcillas terrígenas se encuentran relativamente empobrecidas en estos elementos. El empobrecimiento relativo de Pb, Zn, V, Co y Cr en las arcillas es debido a su movilidad moderadamente elevada. En cambio el empobrecimiento en zirconio se debe precisamente a que este elemento solo

se pierde de la roca por erosión física pero no por erosión química (Riebe et al., 2003). A partir de estas consideraciones y de acuerdo con las correlaciones observadas con la composición mayoritaria, en el sedimento de Ribarroja podemos interpretar la componente 1 como una estima del grado de alteración química de los materiales sedimentados, correspondiendo los valores mas elevados de la misma a aquellas estaciones que presentan un mayor porcentaje de materiales poco alterados, con una presencia significativa de cuarzo y caliza y menor proporción de arcillas. La variabilidad espacial del valor de la componente para el sedimento superficial concuerda con dicha interpretación, observándose los valores mas elevados en la zona de meandros (D, G, E, F, K) y los valores mínimos en las estaciones ubicadas en la presa y en la cola del embalse (A, C, P, M) (Fig. 5).

La componente 2, que explica el 27,8% de la varianza, está determinada por la concentración de níquel (Ni), molibdeno (Mo), galio (Ga), rubidio (Rb) y cobre (Cu) y aparece negativamente correlacionada con la concentración de cromo. Además, presenta una correlación significativa con la concentración de nitrógeno ($r^2 = 0.26$, $p < 0,05$). El hecho de que en el sedimento de Ribarroja Ni y Cu presenten un coeficiente de enriquecimiento relativamente elevado (Tabla 4), y la asociación con la concentración de nitrógeno sugiere que la componente 2 se halla relacionada con un proceso de contaminación orgánica. Los metales de transición, entre ellos Ni y Cu, pueden forman coloides y agregados con arcillas y materia orgánica, lo que explicaría las relaciones observadas. El molibdeno es un elemento escaso que aparece con frecuencia asociado al níquel, por lo que la asociación con la componente 2 no resulta extraña. La interpretación de los resultados relativos a galio y rubidio es menos evidente. La concentración de galio en el sedimento de Ribarroja es del mismo orden que la observada en la corteza terrestre o en arcillas terrígenas (Tabla 4), mientras que la concentración de Rb es netamente superior. Aunque el rubidio es un elemento móvil, se ha descrito un enriquecimiento relativo del sedimento en este elemento por su mayor afinidad de adsorción sobre arcillas, donde desplaza a elementos de menor masa atómica (como por ejemplo potasio) (Kerr et al., 2008). Por todo ello, la componente 2 podría describir básicamente el proceso de formación de complejos entre arcillas-materia orgánica-metales. En este sentido, cabe destacar que la correlación negativa que presenta con el cromo sugiere que este elemento se encuentra ligado a formas inorgánicas en el embalse de Ribarroja. La variabilidad espacial en el sedimento superficial muestra que las estaciones con un mayor valor para esta componente son las ubicadas en la zona central, en especial las estaciones F, K y I, mientras que los menores valores aparecen en las estaciones próximas a la cola C y D (Fig. 5).

Las componentes 3 y 4, que explican respectivamente un 8,4 y 7,4% de la varianza, son de interpretación más compleja y parecen estar relacionadas con la mineralogía de la cuenca. Los elementos que definen ambas componentes, wolframio, torio y cerio, son muy minoritarios y se encuentran a niveles muy bajos en el sedimento de Ribarroja. El peso negativo de la concentración de estroncio con la componente 3 sugiere que se halla relacionada con la naturaleza de los carbonatos presentes, que son materiales con bajas concentraciones de torio.

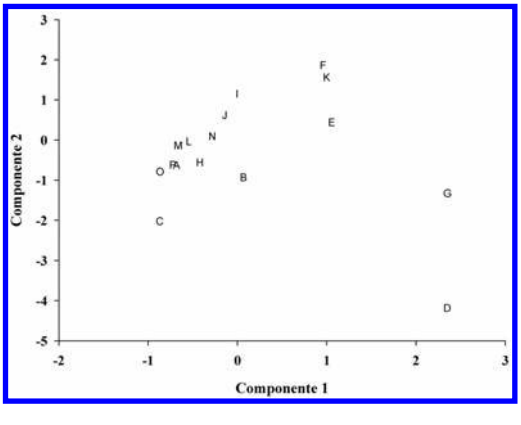

Figura 5. Valores de las dos primeras componentes principales en el sedimento superficial de las distintas estaciones muestreadas a lo largo del embalse.

5 HETEROGENEIDAD ESPACIAL

En términos generales, el sedimento de Ribarroja es relativamente homogéneo. Aunque existen diferencias importantes en cuanto a la cantidad acumulada en distintas áreas del embalse, en conjunto no existen grandes diferencias de tipo cualitativo o en cuanto a la estructura física del sedimento, lo que indica que la procedencia de éste es la misma en todo el embalse. Ahora bien, el análisis detallado de la composición mayoritaria y minoritaria, combinado con el estudio detallado de la estructura física, permite distinguir distintas áreas sedimentarias. Éstas reflejan un proceso de sedimentación selectiva a lo largo del embalse, lo que determina una cierta heterogeneidad espacial del sedimento. A modo de resumen, se describen a continuación dichas áreas destacando sus características más relevantes.

5.1 *Tramo superior*

En el tramo superior se localiza una gran acumulación de sedimento procedente en su mayor parte de los aportes del río Segre. Desde el punto de vista cualitativo se pueden distinguir en esta zona dos áreas claramente diferenciadas: la primera corresponde al área anterior a la entrada del río Segre, próxima a la presa de Mequinenza. En esta zona, el sedimento se caracteriza por una mezcla de materiales muy finos, con una contribución mayoritaria de materiales muy alterados y escasa presencia de materia orgánica y metales asociados. No obstante, al no encontrarse la fracción fina "diluida" por la presencia de materiales de mayor tamaño, la concentración (por unidad de peso seco) de algunos metales como el níquel resulta relativamente elevada.

La segunda área corresponde a la zona con una máxima influencia de los aportes procedentes del río Segre. En éste, el sedimento muestra diversas características diferenciales: está formado por una mezcla de limos, arcillas altamente meteorizadas y arenas poco alteradas. Además, una parte substancial de los limos corresponde a materiales de menor tamaño cohesionados por la presencia de materia orgánica que se encuentra en cantidades moderadas. Paralelamente a la presencia de este tipo de materia orgánica, el sedimento presenta una concentración moderada de cobre, níquel y zinc. En la zona del embalse comprendida entre la entrada del río Segre y el frente de la lengua de sedimento (estación G) el sedimento presenta características compatibles con la sedimentación diferencial de materiales de naturaleza similar a los que se encuentran en el río Segre. La zona más próxima a la entrada del río (estación C) se caracteriza por el predominio de materiales más meteorizados, mientras que, a medida que avanza el frente de sedimento, los materiales menos alterados se hacen más abundantes. Los materiales menos cohesionados, arenas y partículas muy finas, son transportados con mayor facilidad, apareciendo en mayor proporción en la zona frontal de la lengua de sedimento (estación G) y en la zona litoral (estación D). En estas zonas, el sedimento aparece formado por un mezcla heterogénea de materiales, con una baja proporción de limos y una proporción variable de materia orgánica, mayor en la estación más avanzada (G). En ambas áreas, la materia orgánica no aparece asociada a la fracción arcillosa y paralelamente, las concentraciones de metales son bajas.

5.2 *Tramo intermedio*

Este tramo, que comprende la zona de meandros que describe el río Ebro, presenta en conjunto características bastante homogéneas. Se trata de sedimentos de textura limo-arcillosa, en los que los materiales dominantes son los de menor tamaño. La materia orgánica se encuentra asociada a la fracción limosa y su concentración aumenta aguas abajo, observándose las mayores concentraciones de nutrientes en esta zona en la estación K. Los niveles de concentración de metales son moderado-bajos, con valores intermedios entre los observados en el río Segre y en el frente de avance (estación G). Sin embargo, la estación J muestra características muy distintas a las del tramo y parecidas a las de la estación litoral D. Al igual que en ésta, el sedimento se caracteriza por una mezcla heterogénea de arcillas, arenas y un bajo porcentaje de limos; la materia orgánica no aparece asociada a arcillas, pero la concentración de C, N y P es muy superior a la observada en la estación litoral del tramo superior. Del mismo modo, la concentración de metales en esta estación, aunque inferior a la del resto del tramo medio en consonancia con la menor proporción de la fracción limosa, es claramente superior a la de la estación D.

5.3 *Tramo inferior*

El tramo inferior del embalse, desde poco antes de la entrada de río Matarraña hasta la presa de Ribarroja, presenta unas características sumamente homogéneas. Como en el tramo intermedio, el sedimento se caracteriza por una textura limo-arcillosa, pero aunque la proporción de limos se mantiene similar a la del tramo anterior, incrementa ligeramente la proporción de materiales muy finos. En cuanto a la composición mayoritaria, se ajusta a la de materiales muy meteorizados. La entrada del río Matarraña afecta muy poco a la composición del sedimento ubicado aguas abajo y tan solo se manifiesta por un ligero incremento en la asimetría de la distribución del tamaño de partículas tras la entrada del río. Al igual que en el tramo intermedio, una parte de la materia orgánica se encuentra asociada a partículas muy finas. Destaca el incremento gradual en la concentración de nutrientes, nitrógeno y fósforo en el sedimento en la dirección del flujo de agua, alcanzándose las mayores concentraciones poco antes de la presa de Ribarroja (estaciones O y P). La elevada proporción de limos y materia orgánica es paralela a una mayor concentración de metales como níquel, cobre y zinc que presentan concentraciones por unidad de peso seco similares o superiores a las observadas en el río Segre. No obstante, si se tiene en cuenta la menor presencia de material grueso en el tramo final del embalse en relación al río Segre, la concentración de metales en el sedimento del tramo final es ligeramente inferior a la detectada en el río.

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Multi functional small dredging solution for maintenance of deep irrigation reservoirs and hydro power dams

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ABSTRACT: Hydroelectric power is usually regarded as a simple and sustainable technology. That is why it is used all around the world, from developing countries for power and irrigation, to environmentally conscious modern countries for its clean reputation. However, the environment does not leave the construction of a dam unpunished. Its presence does have an impact on the original sediment transport along the river.

The deceleration of the river flow allows suspended particles to settle in front of the obstructing dam. For some projects the natural sediment transport was taken into account in the design. Upstream of the dam a 'sediment collection pit' was created. When filled, a dredge could empty it. However, in most cases the size of the reservoir was chosen big enough that it would take ages to fill up.

Due to global warming, the drainage basin of the river becomes drier and dustier. When rain does fall, the river carries sediment away more easily. Now, a majority of dams face the problem of accelerated siltation and administrations are looking for solutions to extend the lifetime of their infrastructure by dredging the reservoir.

Usually, the bottom profile of the reservoir can vary from very shallow to extremely deep. The dredging equipment provided should cover this whole range, including maintenance in adjacent irrigation channels. Also, variations in water level are very demanding for the configuration of the dredger.

Damen Dredging Equipment developed a range of small modular dredges that accommodates all the demands of a flexible configuration, around the well-known DOP® pump. They come with several modules as ladder, extended ladder and wire rope suspension for the DOP® pump. Also, the DOP® pump can be equipped with various standard suction heads. This approach provides for low-cost, easy to transport units that can be assembled in the most remote places.

1 INTRODUCTION

There are some serious advantages concerning a hydropower or irrigation dam that contribute to a healthy sustainable economy. These will be discussed in a short paragraph to illustrate the interests that participate when looking at the problems that are also caused by such constructions. Also, these will underline the necessity to find a solution to mitigate these problems.

Among others, the sedimentation is the most striking problem in this respect. To remove the sediment there are several options, however, removing it by dredging is the only viable possibility to get it done without any harm to the environment. A dredging tool for this purpose has some specific requirements. These will be investigated shortly.

To develop a new concept, several interesting designs were investigated on their particular usability for implementation for a specialised dam and reservoir maintenance dredge. In the past, several owners and contractors requested or constructed their own special dredges for a specific job. In a short description, their design and their special features are discussed here.

The logical combination of these concepts resulted in the development of the DOP® dredge. Several sizes and operating mode will be presented. Also some examples already build and the experience gained will be discussed.

2 ADVANTAGES OF DAMS

Power generation by a hydropower dam is very attractive for the obvious advantages of this concept over traditional power generation with fossil fuels. There are benefits for the economy, the green house climate control and extra activities provided by the existence of the dam.

2.1 *Economics*

The major advantage of hydroelectricity is elimination of the cost of fuel. The cost of operating a hydroelectric plant is nearly immune to increases in the cost of fossil fuels such as oil, natural gas or coal. Fuel is not required and so it need not be imported. Hydroelectric plants tend to have longer economic lives than fuel-fired generation, with some plants now in service having been built 50 to 100 years ago. Operating labour cost is usually low since plants are automated and have few personnel on site during normal operation.

Where a dam serves multiple purposes, as navigation and irrigation, a hydroelectric plant may be added with relatively low construction cost, providing a useful revenue stream to offset the costs of dam operation. It has been calculated that the sale of electricity from the Three Gorges Dam will cover the construction costs after 5 to 8 years of full generation (Thalmann, 2007).

2.2 *Greenhouse gas emissions*

Since hydroelectric dams do not burn fossil fuels, they do not directly produce carbon dioxide (a greenhouse gas). Although some carbon dioxide is produced during manufacture and construction of the project, this is a tiny fraction of the operating emissions of equivalent fossil-fuel electricity generation.

2.3 *Related activities*

Reservoirs created by hydroelectric projects often provide facilities for water sports, and become tourist attractions in themselves. In some countries, farming fish in the reservoirs is common. Multi-use dams installed for irrigation can support the fish farm with relatively constant water supply. Large hydro dams can control floods, which would otherwise affect people living downstream of the project. When dams create large reservoirs and eliminate rapids, navigation by cargo ships becomes an option to improve transportation.

3 DISADVANTAGES OF DAMS

There are several major disadvantages of power dams, that are well known, albeit maybe less obvious. However, the most important problem for a sustainable exploitation of the dam is the sedimentation in front of the dam.

3.1 *Well known disadvantages*

Hydroelectric projects can be disruptive to surrounding aquatic ecosystems both upstream and downstream of the plant site. For instance, studies have shown that dams along the Atlantic and Pacific coasts of North America have reduced salmon populations by preventing access to spawning grounds upstream, even though most dams in salmon habitat have fish ladders installed.

The reservoirs of power plants in tropical regions may produce substantial amounts of methane and carbon dioxide. This is due to plant material in flooded areas decaying in an anaerobic environment, and forming methane, a very potent greenhouse gas. According to the World Commission on Dams report, where the reservoir is large compared to the generating capacity (less than 100 watts per square metre of surface area) and no clearing of the forests in the area was undertaken prior to impoundment of the reservoir, greenhouse gas emissions from the reservoir may be higher than those of a conventional oil-fired thermal generation plant (Graham-Rowe, 2005).

Another disadvantage of hydroelectric dams is the need to relocate the people living where the reservoirs are planned. In many cases, no amount of compensation can replace ancestral and cultural attachments to places that have emotional value to the displaced population. Additionally, historically and culturally important sites can be flooded and lost. Such problems have arisen at the Three Gorges Dam project in China, the Clyde Dam in New Zealand and the Ilisu Dam in South-eastern Turkey.

Failures of large dams, while rare, are potentially serious. The Banqiao Dam failure in Southern China resulted in the deaths of 171,000 people and left millions homeless. Dams may be subject to enemy bombardment during wartime, sabotage and terrorism. Smaller dams and micro hydro facilities are less vulnerable to these threats. The creation of a dam in a geologically inappropriate location may cause disasters like the one of the Vajont Dam in Italy, where almost 2000 people died, in 1963.

Currently there is a high risk that the Mosul Dam in Iraq might collapse, because it was built on top of a gypsum rock layer. As the gypsum has come into contact with the water from the reservoir, the gypsum slowly dissolves, eventually undermining the structure of the dam (Paley, 2007).

3.2 *Sedimentation*

Over the course of the 20th century, evaporation rates have reduced worldwide, thus making the

climate just drier; this is thought by many to be explained by global dimming (smog and aerosols). As the climate grows warmer and the causes of global dimming are reduced, evaporation will increase again due to warmer oceans. Because the world is a closed system this will cause heavier rainfall, with more erosion of a warmer and drier soil. (Peterson *et al.,* 2002) Figure 1.

The eroded material is carried away by the water running of the catching basin and contributes to the river. There, normal hydraulic processes will form and shape the river in its natural riverbed with the material it carries from the catching basin. So especially with the increased erosion rates, it is vital to investigate the problems related to the sediment transport as it was in its natural state and the impact of a dam construction on this transport. When no adequate measures are taken, the dam will change the geomorphology over time in a way that the negative effects will overcome the implied advantages of the dam as a sustainable energy source.

There are several threats to a sustainable operation of a dam, most notably, the impact on the sediment flux of the original river. The increased water depth and decelerated flow in front of a dam, will make the suspended sediment to settle. The water exiting the dam will have less suspended sediment. On both sides of the dam, this will have an impact. The reservoir in front of the dam will silt up and become smaller Figure 2.

Figure 1. Erosion of catching basin.

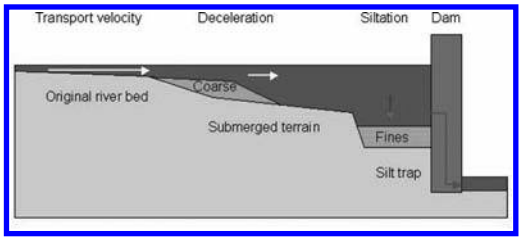

Figure 2. Sedimentation in front of the dam.

In the original riverbed, the velocity of the water is high enough that the sediment is continuously transported downstream. There is a state of equilibrium between incoming and out flowing sediment. However, the course of the river may wander through the landscape with twists and turns.

The material that is eroded from the catching basin or from the wandering riverbed is carried through the river and encounters the slopes of the submerged terrain. Due to the deceleration in velocity, the water flow is not powerful enough anymore to carry the material and it settles on the slopes. Near the entrance of the reservoir a delta like structure forms which consists mostly of the coarser material. Sometimes an artificial deceleration is evoked, by a deep pit or 'silt trap'. This 'silt trap' is taken big enough to fill up only over a very long time, or intended for regular maintenance.

The finer materials take longer to settle, so they are carried further to the deeper part of the reservoir. There the bottom profile is naturally already the deepest part where the flow is almost zero. Sometimes a silt trap is placed there to get all the particles out of the water, before the water enters the turbines. Anyway eventually the reservoir in front of the dam can become completely filled with sediment Figure 3.

This problem has not been recognised in the very early dams, but gradually, the engineers were able to predict the sedimentation behaviour. Usually, the sedimentation follows a first order response. The reservoir will gradually decrease in volume and eventually will be completely filled Figure 4.

Generation of hydroelectric power changes the downstream river environment also. Water after the dam usually contains very little suspended sediment, which can lead to scouring of riverbeds

Figure 3. Basin 100% filled up.

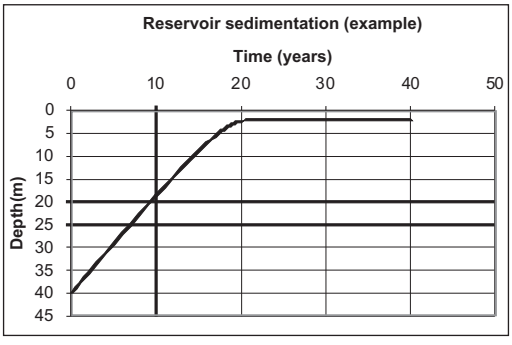

Figure 4. Reservoir capacity over time.

Figure 5. Water inlet flushing.

and loss of riverbanks. Also this problem is a concern for a sustainable maintenance of the area behind the dam.

When the water of the reservoir is not only used for power generation, but also for irrigation, there is another problem with the silt. It also tends to clog the small canals, that branch from the reservoir to the agricultural area. This requires regular maintenance of a dredge small enough to enter those canals.

3.3 *Remedial flushing*

The siltation problem was recognised early and the majority of the dams have some special precautions to catch the settling material before it could reach the water inlets of the turbines. Usually the area in front of the dam was deepened to act as a silt trap. The velocity was decelerated so much that virtually every particle settles in front of the dam. The volume of the silt trap was chosen that it would take ages to fill up and no maintenance should be necessary. However, due to the increased erosion, the sand traps silted up pretty much. The bottom levels become so high, that the water inlets were taking sand in, endangering the turbines.

In order to clear the water inlets, the flushing gates in the dam are opened Figure 5. The increased velocity of the water in front of the dam resuspends the sediment and transports it below stream of the dam. Exactly the latter is problematic, as all the sediment that has accumulated over several years is released at once. Due to the very high sediment load, all marine life in the river is flushed away or buried alive.

Due to the regulations concerning the flushing of dams and that most dams were designed to work that way, a lot of dams are not allowed to operate anymore. In some cases more than 50% is not operating for its full capacity. So it is compelling to find a solution to clean the water inlets from the accumulated sediment. (Remini *et al.,* 2007).

A much better option would be to remove the sediment in front of the dam by a dredge and bring the material on shore. That way, the delivered sand can be used for construction or filling. When the material is not required, it can be bypassed around the dam and deposited in the river again. It will be carried away by the flow. As the sediment load is continuous and at a much lower level than flushing. It might be a good solution for disposing of the material.

4 DREDGE DESIGN REQUIREMENTS

In order to design a dredging solution to sedimentation problem of the deep hydropower and irrigation reservoirs, the maintenance problem has to be investigated in detail. We will look into the issues related to operations flexibility, deep dredging requirements, transport limits, service requirements and costs.

4.1 *Flexible operation*

The reservoir can be very deep directly in front of the dam. It gradually shallows to nothing where the water enters the reservoir. Waterway and irrigation channels branching from the main reservoir tend to be very shallow and small. Figure 6 So in order to operate at the reservoir, the concept has to be flexible enough to adapt to the various tasks. A modular design that can be easily changed to meet the specific operating requirements might be a good solution.

4.1.1 *Waterway maintenance*

The original river flowing into the reservoir might be used for navigation. Usually there is some

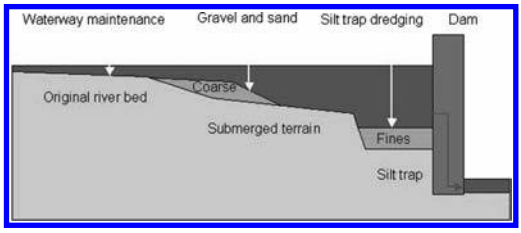

Figure 6. Various maintenance operations.

infrastructure around the original river that has to be kept in order, also. It would be a great benefit if the dredge operating on the reservoir would also be able to do the maintenance dredging in those adjacent waterways.

For the maintenance of the waterways around the reservoir and the original river, a dredge with only a relative small dredging depth is required, however, it should be able to excavate according to a required profile, possibly in the original bottom material. This requires that the dredge is equipped with a cutter head and spuds.

4.1.2 *Sand and gravel dredging*

Where the river flows into the reservoir, a natural delta like structure forms with the coarsest material from the transported sediment. Usually this constitutes of gravel and sand, which could be used beneficially for other purposes. To excavate this material, the dredge would need a cutter head and should be able to work on deeper water. So a Christmas tree is a logical choice for positioning the dredge. As the final profile of the delivered reservoir bottom is less important, the exact positioning of the cutter head is not required.

The heavier sand and gravel, is harder to dredge with an inboard dredge pump. Logically, a submerged dredge pump would be good concept for this operating mode.

4.1.3 *Silt trap dredging*

Directly in front of the dam, the reservoir is usually deeper. In case there is a sand trap, the reservoir is deepened intentionally. As at this location, the material is very fine, it tends to flow in a gravity current to a lower location. This is an excellent material to be excavated in a deep dredging mode, where an initial suction pit is created and the material flows from active banks to the suction pit. Moreover, the deeper the pit, the more productive the banks are.

The exact bottom profile in front of the dam is absolutely of no concern, as long as the turbine inlets are positively free of incoming sediment.

4.2 *Deep dredging requirements*

For the maintenance of the area in front of the dam, near the water inlets, the material has to be dredged from a deep level. An inboard dredge pump can't do this, so naturally a submerged dredge pump has to be chosen.

Another problem encountered in reservoir dredging is, that the dams are located at high altitude. This means that the available air pressure is lower than at sea level. This would cause major cavitation problems in an inboard dredge pump also Figure 7.

When p_{atm} decreases from 100 kPa at sea level, to 70 kPa at 2000 m altitude, you lose 30 kPa of usable vacuum pressure. Your available $NPSH_A$ can lower dangerously close to the required $NPSH_R$ with a high risk to cavitation. In this example, you are virtually working on a dredging depth of 3 m deeper, than indicated.

So even at shallower working depths, there is a possibility to experience cavitation. It is therefore important to have a short suction line and a small difference between dredging depth and pump level. This is another reason to look for a concept with a submerged dredge pump.

When there was not enough effort put in clearing the area that later became the reservoir, the original vegetation will decompose in a sometimes deoxygenated environment. This will result in serious fermentation gasses that become trapped in the new sediment that settles on top. When the gas is released in bubbles from the bottom through the disturbances of the dredging operation, they will be sucked into the suction pipe. In the suction pipe, the pressure drops and the gas, in the bubbles, expands. As the pressure in the suction side of the dredge pumps expands to virtually zero, the bubbles occupy a large volume of the inside of the impeller and the pump will not be able to develop a continuous flow. Effectively the pump stalls in a similar fashion as cavitation, where vapour bubbles block

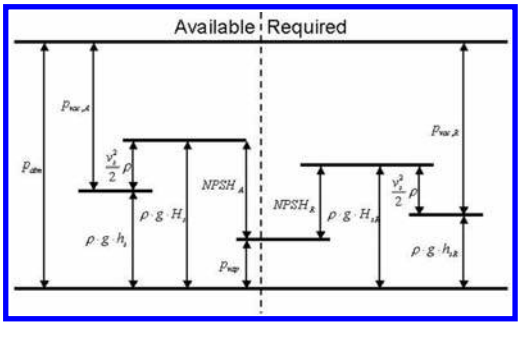

Figure 7. NPSH boundary conditions.

the pump. To overcome the gas bubble related problems in the dredge pump; the pressure on the suction side of the dredge pump should be as high as possible. The easiest solution would be to apply a submerged dredge pump.

4.3 *Transportability*

Reservoirs and dams tend be located in remote areas and rough terrain, usually inaccessible for heavy construction equipment. So the whole dredge has to be small or comprised of small modules that can be assembled together without heavy construction equipment.

Maximum module weight should be less than 10 ton and the outside dimensions should be less than a 40' container unit.

Preferably, the dredge should be assembled from as few modules as possible and only a few connections between the modules as necessary.

The total weight of the assembled pontoons in their smallest configuration should be low enough that the pontoons can be put together on the shore and pulled or pushed in the water so no heavy crane is necessary.

4.4 *Service*

The companies operating the dredge might not be acquainted with normal dredging components and marine equipment. It is probably easier for them to use a technology that is much more available in the construction industry. This will ensure that when something breaks down, the parts might be available at the next truck maintenance centre, instead of having it flown in from a far away coastal city.

5 CONCEPT INVESTIGATION

There are several special dredges that are interesting for examining their special concepts on their suitability for a remedial cleanup project of a reservoir.

5.1 *Laurum*

Although the 'Laurum' was not exactly intended for reservoirs, the versatility of the design is very interesting in this regard.

The Laurum was commissioned by Ballast Nedam Dredging, with the explicit order to have a variable working depth and an easy to transport modular hull construction. All hull sections are container sized for minimum transport effort (Adams *et al.,* 1994).

Since the 'Laurum' has a great working depth range of 3 to 55 m, and it is inefficient to use a

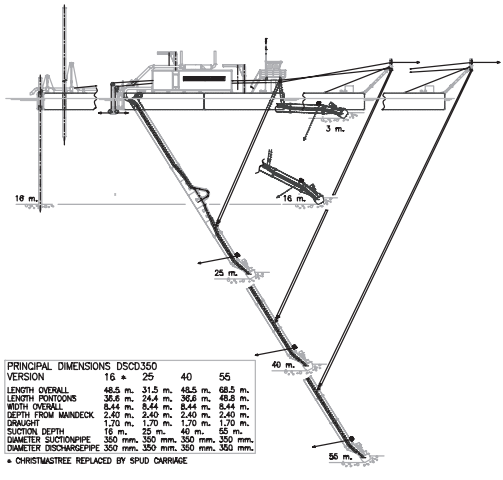

Figure 8. Operating modes 'Laurum'.

48.5 meter long cutter ladder (needed for a working depth of 40 m) at working depths of 3 to 20 m, it was clear the dredger and cutter ladder should consist of dismountable modules, so a shorter version could be created by taking away pontoon and cutter ladder sections Figure 8.

Ballast Nedam Dredging however decided to go one step further, and required the modules to have standard container sizes, supplied with standard container corner castings. Because of the standardized dimensions of the modules, transport by road, water or rail is trouble-free and by far the most economic means of conveyance.

This dredge has been a benchmark for later models and often referred to in the dredging industry. Its modular design is well known for its excellence in flexibility, deep dredging and easy transport.

5.2 *La Mahuna*

The Cutter Suction Dredge 'La Mahouna' was ordered by the company Hydrodragage from Algeria to operate in a reservoir Figure 9.

The 'La Mahuna is equipped with a submersible DOP® pump for dredging at depths ranging from depths 16 to 40 meters. The bottom of the reservoir was covered with decomposing organic material that contributed in situ gas. The DOP® pump did not have any problem with the high gas content. To bring the mixture a reasonable distance to a handling area, a booster pump on deck delivered extra pressure to the mixture.

The hydraulic power unit for the DOP® pump and the diesel engine for the booster pump were installed in container sized housings. The pontoons were standard container size modules, also.

Figure 9. 'La Mahuna'.

Figure 10. 'Groβglockner'.

Key words for this design are: deep dredging and transport.

5.3 *Groβglockner*

The 'Groβglockner' has been build by the contractor Dutch Dredging, especially for reservoir maintenance. Therefore it has some special features that have to be discussed in this context Figure 10.

The pipe is lifted and brought down into the material by a hoisting installation. To prevent trees, wood, roots and other debris to enter the pump a special suction head with a strainer is developed. At the end of the suction head a cone is installed to push possible debris aside. With this the production losses are minimised and the suction inlet can enter deeper into the material. Another advantage of the vertical pipe is that the dredge head can follow the soil even in fast changing water levels and thus optimising production. Last but not least it can easily be pulled without damage out of material fallen on the pipe in case of slides.

The DOP® pump brings the material up through a flexible pipeline and deposits the mixture through a strainer in an open tank on the starboard quarterdeck. This tank contains the other DOP® pump, which transports the mixture to the delivery pipeline (towards a booster station or dumping area).

When this dredge has to be characterised to its main design features, the following key words are applicable: deep dredging and transport.

5.4 *Balderhaar*

The sand production dredge 'Balderhaar' is an excellent example of a successful deep dredge using only a DOP® pump suspended on a gantry. The dredging contractor MegaMix needed a small dredge that was capable of dredging at great depths.

Figure 11. 'Balderhaar'.

As the DOP® pump is an excellent deep dredging tool, the dredge delivers a high concentration and good production Figure 11.

The components were very simple and easily mobilised. Standard or marginally adapted power packs and equipment were used for the installation. This resulted in low investment costs.

The resulting dredge scores very good at the aspects: deep dredging, transport and service.

6 SOLUTION

Taking all the major requirements: flexible, deep dredging, transport, and service into consideration, we designed a modular dredge incorporating the innovative concepts of the above-mentioned examples. The resulting dredge is a multi functional dredging tool. It is either a cutter suction dredge or a plain suction dredge which is equipped with a DOP® submersible dredge pump. The DOP® pump can be attached to the boom of a crane or it can

be lowered by a hoisting cable. The DOP® pump can also be provided with a cutter unit and than attached to a fixed ladder.

The flexible modular design of the DOP® dredge has many advantages:

- The dredge can be transported and put together easily as it consists of relatively small components.
- The dredge can be supplied as an assembly package (for example pontoons built locally to a Damen Dredging Equipment design). This concept requires low investments and low transport costs.
- The DOP® pump is lowered to the required working depth. The submersed pump creates an optimal vacuum and guarantees high solids content compared to an inboard dredge pump.
- The DOP® pump can be dismantled easily to be deployed elsewhere, for instance on an excavator.
- On deck, a similar pump can be placed working as a booster station. The pumps would need identical parts and are therefore easy to service.

6.1 *Excelling points*

6.1.1 *Flexible operation*

The operations of a dredge working on a reservoir can vary dramatically. From the small canals and river at the beginning to a deep dredging operation in front of the dam Figure 12.

By choosing a DOP® pump as the prime mover, all operating modes guarantee a good concentration and a high production. In the smallest configuration, the DOP® pump is attached directly to a short ladder. For the increased dredging depth, an extension of the ladder is used. Extra pontoons are added to handle the extra length. For the real deep dredging operation, the DOP® pump is disconnected from the ladder and suspended directly to the gantry. The discharge of the DOP® pump is connected to a riser.

Using standardised pontoons, the size of the dredge can be adapted to the operating mode required and still maintain enough buoyancy and control oer the ladder and the DOP® pump.

6.1.2 *Deep dredging*

As explained before, for deep dredging it is vital to use a submerged dredge pump. Using a DOP® pump is especially effective as it has the shortest possible suction line and the closest presence of the dredge pump to the suction mouth. Usually a deep dredging installation still works with a fixed submerged dredge pump and a long suction pipe. If the DOP® pump is suspended from the gantry, the dredging process is very similar to deep dredging Figure 13.

As seen in the 'Balderhaar' example this is a concept that works very well. One of the most important components in this concept is the riser. The riser connects the DOP® pump with the dredge. This riser has been specially developed for the DOP® dredges, so it will also carry the hydraulic power lines and the jet water hoses to the DOP® pump and still have enough buoyancy for the varying working depths.

6.1.3 *Transportability*

With the design of the pontoon components, the boundary conditions for transport as individual weight and dimensions were fulfilled. As the pontoons are only used as floatation, the pontoons can be assembled ashore and pushed in the water at an early stage of mobilisation. Later, the important equipment as power pack, winch platform and operating cabin are placed on deck and connected to each other. No system connections between those three components and the pontoons are necessary. All other components of the dredge can be fitted with the dredge already in the water, so minimal lifting capacity for mobilisation is necessary.

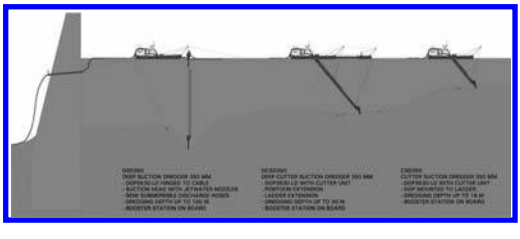

Figure 12. Flexible operating modes.

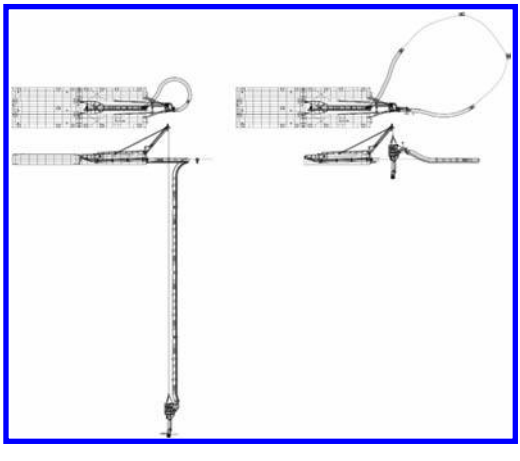

Figure 13. DOP® dredge with riser.

The pontoons and power pack of the DOP® dredge 350 are container sized. The rest of the components fit in the containers themselves. For the smaller types, the whole dredge fits in three 40' containers Figure 14.

6.1.4 *Service*

As all the main components of the dredge are built with standard equipment, the spare parts are readily available. Engine and hydraulics can be serviced locally and the components of the DOP® pump through a worldwide network.

6.2 *Types*

With the size of the reservoir and the sedimentation rate, different sizes of dredges are required. There are three different designs each available with the following specifications Table 1.

6.3 *Standard equipment*

The DOP® dredge can be delivered in all operating versions separate or as a complete package where all the components for ladder, ladder extension and riser are supplied together. So the total delivery of the dredge can be well adjusted to the specific needs of the client. Even clients not related to the administration of a reservoir are very interested in the flexible model composition of the DOP® dredge.

Figure 14. DOP® dredge 200 packed in three containers.

The basic execution of the DOP® dredge is equipped with the following standard equipment.

6.3.1 *Hull and superstructure*

The dredge is dismountable in main pontoon and four side pontoons, operating cabin, winch platform and gantry. Depending of the version a cutter ladder, ladder extension and/or riser are added. All modules are small enough to permit easy transportation by road, rail or ship to nearly any location. The pontoons are connected with a heavy duty coupling system making (dis)assembly on land or afloat possible in a very short time and an easy way.

On for and aft corners of the pontoon, single bollards are provided for mooring and towing. The deck is surrounded with removable railing made of stanchions and stainless steel wires.

The hull and superstructure are protected by a marine coating system and cathodic for inland and seawater use.

6.3.2 *Operating cabin*

The dredge is operated from a very spacious ErgoCab® operating cabin. The cabin is ergonomically designed according to the latest insights. The standard cabin is equipped with air-conditioning and heating. To minimise vibration and noise levels, the cabin is mounted on shock absorbers. Also, the cabin is constructed of double steel walls and well insulated.

All controls are built in ergonomic designed control panels and the operator is seated in a dredge master chair.

Dark tinted glasses all around of which one can be opened, providing excellent view of all essential deck equipment.

6.3.3 *Dredging equipment*

A DOP® pump is used as the prime mover of the dredge. This DOP® pump is a high efficiency dredge pump, built up with NiHard4 wearing plates and pump casing and bainitic nodular impeller. The shaft is sealed using a maintenance free mechanical seal.

Cutter shaft supported by roller bearings mounted in an oil-filled casing. The slow running hydraulic cutter motor is well protected in the cutter unit. The cutter is equipped with replaceable wear resistant serrated edges or knives.

Table 1. Descriptive title of table.

Type	Pump	Power	Working depth	Weight	Mixture capacity
200	DOP 1815	225 kW	up to 25 m	22 ton	300–600 m^3/h
250	DOP 2320	310 kW	up to 25 m	27 ton	500–1000 m^3/h
350	DOP 3530LD	450 kW	up to 40 m	45 ton	1200–2000 m^3/h

The side wire and aft winches are mounted on a winch platform.

6.3.4 *Engines and hydraulics*
Latest model marine engines, complying IMO regulations are installed in a power pack with a noise insulated housing. There is a fan driven air-cooling system in the engine housing.

A hydraulic pump and motor circuit drive the dredge pump. Variable axial piston pumps driven by the diesel engine operate all hydraulic motors. The hydraulic system includes: stainless steel tank, all required electric operated valves, filters, gauges etc.

6.3.5 *Electrical installation*
The electrical installation of the dredge is 24DC voltage only. The diesel engine has its own starting battery. The alternator of the diesel engine charges the set. The power distribution board is placed in the power pack.

Communication between operating cabin and power pack is done over a PLC bus system. Electric connections between cabin and power pack with multi-pin sockets for quick (dis)assembly without the possibility of wrong connections.

6.3.6 *Auxiliaries*
The following loose equipment will be supplied with the dredge:

- Mooring lines
- Life saving equipment
- Navigation equipment
- Set of tools, including impeller hook
- Boatswain's inventory
- Start up spare parts for initial assembly

6.4 *Optional equipment*

The standard version of the DOP® dredge is well equipped already. However, there are several extra options available that can be delivered with the dredge.

- Navigation lights
- Suction pipe vacuum relief valve
- Production instrumentation
- Revolution counter dredge pump
- Dredge profile indicator
- Positioning and survey systems

6.5 *Examples*

Based on the above-mentioned concepts, the first standard DOP® dredges have been built and tested on their performance.

The smallest of the range is the DOP® dredge 200 Figure 15.

The largest in the range is the DOP® dredge 350 Figure 16.

Figure 15. DOP® dredge 200.

Figure 16. DOP® dredge 350.

Both dredges are performing to specification and operating to full customer satisfaction.

7 SUMMARY

There is a lot of work to be done on the neglected maintenance of reservoirs, in order to continue a sustainable operation. If the maintenance is further postponed, the costs of remedial operations will only increase further. A quick and sound solution for this problem is required.

The Damen DOP® dredge is a versatile toolbox for the maintenance of hydroelectric power dams and irrigation projects. The possibility to dredge the annoying sediment and even put it to beneficial use, greatly contributes to the sustainable character of a hydropower plant.

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K Nearest Neighbor approach to estimating the fall velocity of sediment particles

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ABSTRACT: Drop velocity estimating is one of the most important issues in sediment sciences, which helps in dealing with modeling sediment transport. While, many conceptual, empirical, and statistical methods have been proposed for estimating and forecasting, rarely a single model (an experimental equation) is found as the best estimating model in all hydraulic and sediment conditions experienced in sediment modeling. A solution for this problem is to use data fusion approach to combine advantages of individual models to improve an estimation skill. As a matter of fact, data fusion provides better estimations than could be achieved by the use of individual estimation models. This paper presents a comparative assessment of five different methods of data fusion by applying them in six laboratory data sets. Furthermore, a new statistical method based on the non-parametric K- nearest neighbor model is presented as an alternative method for data fusion. Results of data fusion approach is thoroughly analyzed and discussed. The results demonstrate that the use of proposed data fusion model could significantly improve the accuracy and precision of estimations in comparison with the use of singe models or even conventional data fusion methods.

1 INTRODUCTION

One the most important issues in the sediment contest is developing suitable estimate models for the purpose of fall velocity estimation. The qualitative analysis of sediment transports in river engineering problems, such as sedimentation in river courses and morphological changes of river banks, designing the settling basins of water conveyance networks, and sedimentation of dam reservoirs, needs to use a suitable relation to estimate the terminal fall velocity, sometimes called settling velocity, of sediment particles. The terminal fall velocity of a particle is the particle downward velocity in a low dense fluid at equilibrium in which the vector sum of the gravity force, buoyancy force and fluid drag force being equal to zero.

Fall velocity of a particle, depends on the density and viscosity of the fluid, and the density, size, shape, spherically, and the surface texture of the particle. Many attempts to predict the particle fall velocity have been carried out by researches, started by Stokes in 1851 [cited in Graf 1971] and followed by Oseen (1927), Rubby (1933), Rouse (1938), Interagency Committee (1957), Zanke (1977), Yallin (1977), Hallermier (1981), Dietrich (1982), Van Rijn (1989), Concharov [cited in Ibad-zadeh 1992], Julien (1995), Cheng (1997), Jimenez and Madsen (2003), Brown and Lawler (2003), She et al. (2005), and Wu and Wang (2006) among others, who developed empirical or semi-empirical relations for estimating the settling velocity of sediment particles.

Most of above mentioned investigations, however, have some limitations when it comes to applying them to engineering works. For instance, the relations developed by Stokes [cited in Graf 1971], Rouse (1938), Brown and Lawler (2003), are applicable only to spherical particles. Even for spherical particles, the analytical solution of Stokes is only applicable for Reynolds number less than 1 and there is no analytical solution to predict the fall velocity of natural particles. In the absence of such a solution, some laboratory investigations have been conducted to provide design curves to predict the fall velocity based solely on the diameter of standard particles (e.g. Rubby 1933; Graf 1971; Baba and Simons 1981 among others). A family curves, also, were provided to predict the effects of other particle characteristics on fall velocity; e.g. Alger and Simons (1968), Komar and Reimers (1978) among others.

In a useful attempt, the US Interagency Committee on Water Resources summarized the data

obtained by several researchers, and published a graphical relation to estimate drag coefficient, and consequently, to calculate settling velocity [cited in Wu and Wang 2006]. This graph, however, includes a series of curves and tables, and several interpolations must be conducted to obtain the answer, making it inconvenient to use.

Recently, Wu and Wang (2006) have re-evaluated the relation of US Interagency Committee using a wide range of data, and using the equation proposed by Cheng [cited in Wu and Wang 2006], introduced an explicit mathematical expression for the settling velocity of natural sediment particles. Wu and Wang (2006) reported that by considering the effects of viscosity and Corey shape factor, their formula has a relative mean error of 9.1% which decrease to a relative mean error of 6.8% when the effects of Corey shape factor are neglected. They concluded that their relation performed better than nine existing formula in the literature.

Data fusion is an emerging area of research that covers a broad spectrum of application areas ranging from ocean surveillance, strategic warning, and medical diagnosis (Hall 1992). The principal objective of data fusion, which is the process of combining or amalgamating information from multiple sensors and/or data sources, is to provide a solution that is either more accurate according to some measure of evaluation, or allows one to make additional inferences above and beyond those that could be achieved through the use of single source data alone (Dasarathy 1997).

Fusion is often a mix of recognized procedures that are applied using either (i) modular strategies in which different models are developed to perform different modeling operations and then coupled together (ii) ensemble strategies in which different models are developed to perform similar modeling operations (Hall 1992). Data fusion researches are divided into two board groups. The first takes the view that data fusion is the amalgamation of raw information to produce an output while the second advocate a more generalized view of data fusion in which both raw and processed information can be fused into useful outputs including higher level decision. Recently, researches such as See and Abarhart (2001), Abarhart and See (2002), and Shu and Burn (2004) have used model-fusion approaches in hydrological engineering. Shu and Burn (2004) applied data fusion method to combine individual ANN models in order to enhance the final estimation.

This paper provides a comparative assessment of five general methods of data fusion in fall velocity estimation. Furthermore, a new statistical method based on the non-parametric *K*-nearest neighbor method is proposed for the purpose of data fusion. These methods are compared with experimental equations which are presented and utilized by researchers in recent decades. Application of these methods is tested in laboratory data and their results are analyzed and compared with each other.

2 DATA FUSION METHODS

The general equation of an estimatingting model is

$$\hat{y}_i = f(X_i) + \varepsilon_i \qquad i = 1, ..., n \tag{1}$$

where X = vector of predictors, y = estimation variable, ε = model error and n = number of observation data. In the case of using multiple models to estimate y, and considering similar predictors, equation 1 is changed to the following matrix form

$$\left[\bar{Y}_i\right] = \begin{bmatrix} \hat{y}_{i1} \\ \hat{y}_{i2} \\ \vdots \\ \hat{y}_{im} \end{bmatrix} = \begin{bmatrix} f_1(X_i) \\ f_2(X_i) \\ \vdots \\ f_m(X_i) \end{bmatrix} + \begin{bmatrix} \varepsilon_{i1} \\ \varepsilon_{i2} \\ \vdots \\ \varepsilon_{im} \end{bmatrix} \qquad i = 1, ..., n \tag{2}$$

where m = number of estimation models used to estimate y. $[\hat{Y}]$ = Matrix of estimations of y provided by different individual models. Using the data fusion approach, $[\hat{Y}]$ is sum up to an unique estimation of $\hat{y}$. The following paragraphs describe some of general methods of data fusion, presented and implemented by different researchers.

2.1 *Simple and Weighted Averaging Methods (DF1 and DF2)*

Linear combination of the outputs of ensemble members is one of the most popular approaches for combining different outputs. A single output can be created from the outputs of a set of models via simple averaging, or a weighted average that considers the relative performance of each model. Combining using simple average is defined as:

$$\hat{y}_i = 1/m \left(\sum_{j=1}^{m} \hat{y}_{ij} \right) \qquad i = 1, ..., n \tag{3}$$

Despite its simplicity, the method suffers from the problem of considering equal weights for individual models, where their estimating value might be significantly different. To overcome this shortcoming, the method of weighted averaging also known as stacking might be used. This method is presented by the following equation:

$$\hat{y}_i = \sum_{j=1}^{m} c_j \hat{y}_{ij} \qquad i = 1, ..., n \tag{4}$$

where, c = the weight of each individual models. Under stacking an additional model is used to learn how to combine the model by tuning the c weights over the calibration data. The stacking algorithm developed by Shu and Burn (2004) suggests minimizing the following function to derive c weights.

$$w = \sum_{i=1}^{n} \left[\frac{y_i \sum_{j=1}^{m} c_j \hat{y}_{ij}}{y_i} \right]^2 \quad ck > 0 \tag{5}$$

The stacking method uses constant weights over the period of calibration data set which it reduces the flexibility of the method in facing with the different situation that a system might experience during its operation.

2.2 *Linear Multiple Regression Model (DF3)*

Linear Multiple regression is a multivariate method of estimating, which estimates unknown coefficients of predictors by performing a least squares fit. In this study, the Linear form of Multiple Regressions (*LMR*) is applied as a estimating method in the following form:

$$y = a_1 x_1 + a_2 x_2 + \cdots + a_p x_p + c \tag{6}$$

where,

a_1 to a_p = p predictors of dependent variable y,
x_1 *to* x_p = coefficients of predictors,
c = constant of the model.

2.3 *Artificial Neural Networks (DF4)*

Empirical models, particularly Artificial Neural Networks (ANNs) are known as powerful tools for function mapping. See and Abarhart (2001) have suggested the use of ANNs as a data fusion method. The general form of this method is

$$\hat{y}_i = g([\hat{Y}]')[\hat{Y}] = \begin{bmatrix} \hat{y}_{i1} \\ \hat{y}_{i2} \\ \vdots \\ \hat{y}_{im} \end{bmatrix}; \qquad i = 1, \ldots, n \tag{7}$$

where g is non-linear function which maps outputs of different individual estimation models to a single output of $\hat{y}_i$ using an ANN model. This function is trained by pairs of through the calibration data se. Among different ANN models, feed forward back propagation ANN could be most appropriate for data fusion purposes. Like most of empirical methods, this method suffers from the lack of statistical sense in the mechanism of data processing.

2.4 *Adaptive neuro-fuzzy inference system (ANFIS) (DF5)*

The basic structure of the type of fuzzy inference system seen thus far is a model that maps input characteristics to input membership functions, input membership function to rules, rules to a set of output characteristics, output characteristics to output membership functions, and the output membership function to a single-valued output or a decision associated with the output. You have considered only fixed membership functions that were chosen arbitrarily. You have applied fuzzy inference to only modeling systems whose rule structure is essentially predetermined by the user's interpretation of the characteristics of the variables in the model.

The neuro-adaptive learning method works similarly to that of neural networks. Neuro-adaptive learning techniques provide a method for the fuzzy modeling procedure to *learn* information about a data set. Fuzzy Logic computes the membership function parameters that best allow the associated fuzzy inference system to track the given input/output data. The acronym ANFIS derives its name from *adaptive neuro-fuzzy inference system*. Using a given input/output data set, ANFIS constructs a Fuzzy Inference System (FIS) whose membership function parameters are tuned (adjusted) using either a backpropagation algorithm alone or in combination with a least squares type of method. This adjustment allows fuzzy systems to learn from the data they are modeling.

3 PROPOSED DATA FUSION METHOD BASED ON K-NN (DF6)

The recognition of the nonlinearity of the underlying dynamics of hydrological processes, have spurred the growth of nonparametric methods. Nonparametric estimation of probability densities and regression functions are pursued through weighted local averages of the dependent variable. This is the foundation for nearest neighbor methods. *K*-Nearest Neighbor (*K*-NN) methods use the similarity (neighborhood) between observations of predictors and similar sets of historical observations (successors) to obtain the best estimate for a dependent variable[Karlsson and Yakowitz (1987), Lall and Sharma (1996)].

For each state t, let $[X_j]$ (j = 1, 2, 3, …, number of observed data) be an P-dimensional feature vector of past records. A feature vector is a vector

that summarizes the past history in a smaller-dimension vector of observations that contains most of information relevant to the estimation. To estimate depend variable in state t, y_t, the K-NN method imposes a metric on feature vectors to find the set of K past nearest neighbors of $[X_j]$.

The K vectors of past observations are obtained that have the minimum norm among all candidates. The most widely used metric to identify neighbors is the Euclidean norm, which, for a P-dimensional feature vector, is calculated as equation (8):

$$Dis_{tj} = \sqrt{w_1(x_{1t} x_{1j})^2 + w_2(x_{2t} x_{2j})^2 \ldots + w_p(x_{pt} x_{pj})^2}$$

where, w = the weight of predictors in calculating the distance between the current value of predictors and the neighbors. The forecast is then obtained as a weighted average of the nearest neighbors, such that greater weight is assigned to the nearer neighbors. A kernel function proposed by Lall and Sharma (1996) defines the weights, which leads to a K-NN regression estimate of:

$$y_t = \frac{\sum_{i=1}^{K}(1/i)y_i}{\sum_{i=1}^{K}(1/i)} \tag{9}$$

where i is the order of the nearest neighbors in which the nearest have the lowest order (i = 1 to K), and, y_i is the magnitude of nearest neighbor i. The weights and the number of the neighbors that produce the lowest mean square error of the estimating are found by computing the error for all combinations of weights and K values (from 1 to $n - 1$) through cross-validation of the calibration data set. Schematic of K-NN estimating algorithm are shown in Figure 1. Using the concept of K-NN, an algorithmic procedure is proposed for data fusion as follows:

1. Use m individual estimating models to produce $n \times m$ estimations, where n is the number of observed data used for calibration.

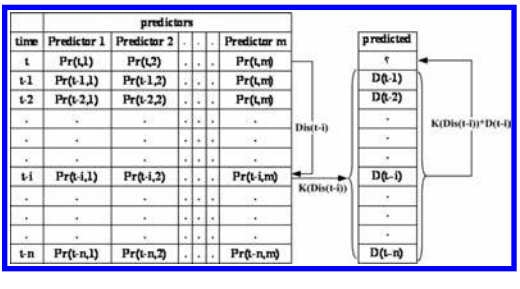

Figure 1. Schematic of K-NN estimating algorithm.

2. Evaluate individual estimating models in all n forecast experience. Compute matrix of $[A] = [a_{ij}]_{n\times m}$, where $a_{ij} = 1$ if mth model results in the best estima-tion during ith experience; otherwise $a_{ij} = 0$.
3. At the present state, t, compute m estimations of y_t, using m individual estimation models and development.

$$\left[\hat{Y}\right] = \begin{bmatrix} \hat{y}_{i1} \\ \hat{y}_{i2} \\ \vdots \\ \hat{y}_{im} \end{bmatrix}$$

4. Compute

$$[F] = [A] \times \left[\hat{Y}\right] = \begin{bmatrix} f_1 \\ f_2 \\ \vdots \\ f_n \end{bmatrix}$$

5. Use equation 8 to find distance between present state predictors, $[X_t]$ and the historical predictors, $[X_j]$ Determine the nearest neighbors from n sets of ob-served data, such that the smaller distance is as-signed to the nearest neighbor.
6. Estimate the dependent variable by the following equation

$$\hat{y}_r = \frac{1}{\sum_{i=1}^{K} 1/i} \sum_{i=1}^{K} (1/i) f_i \tag{10}$$

where i = the order of the nearest neighbors in which the nearest have the lowest order (i = 1 to K), K = number of nearest neighbours (specified by user), and = the magnitude of nearest neighbor i.

4 DEVELOPMENT OF EXPERIMENTAL EQUATIONS AS INDIVIDUAL MODELS

This section describes briefly the experimental equations used as individual models in this case. In 1851 Stokes by using Navier-Stokes equations, along with continuity equation expressed in polar coordinates, investigated the coefficient of drag applies by fluid flow upon a spherical particle (Graf 1971). Based on Stokes' results, the fall velocity of spherical particles in region of particle Reynolds number (R_e) less than 1, can be calculated using (Cheng 1997):

$$w = \frac{1}{18}\frac{g(s-1)d^2}{\nu} \tag{11}$$

In which w = particle fall velocity in m/s, g = acceleration due to gravity in m/s^2, d = particle diameter in m, v = kinematic viscosity in m^2/s, and s = relative density (ρ_s/ρ) where ρ_s and ρ = the density of sediment particle and fluid in t/m^3, respectively.

For natural sediment particles, many researchers have attempted to develop similar equation. Due to extensive variation of natural particles' geometry, however, there has been a little success in this regard, so that a large number of different equations, each of which can only be applied to a limit range of sediment and fluid conditions, have been developed. In this research, famous relations of fall velocity introduced from 1933 to 2006, have been collected and their advantages and limitations have been investigated. Based on this, for the purpose of comparison, six relations have been chosen. Output of these six experimental equations (EE1 to EE6) is input of data fusion models; it means that the results of these experimental equations are combined by data fusion models to obtain achievements. (See Table 1)

5 APPLICATION OF THE MODELS

The schematic of the procedure for data fusion, experimental equations as individual models used in the procedure, and the methods of data fusion are shown in Figure 2. As shown in Figure 2, data fusion methods 4, 5 and 6 use hydraulic predictors beside the output of individual models (experimental equations) to develop a combined estimation.

Three statistics are used for comparison of the results of each model as well each fusion method. The Root Sum Square Error (RSSE) is defined as:

$$RSSE = \sqrt{\sum_{i}^{n}(obs_i - for_i)^2} \times 100 \tag{12}$$

where obs_i is observed data at state i, for_i is estimated data at state i and n is the number of estimated values. Percentage volume error (%VE) is defined as:

$$\%VE = \frac{\sum_{i}^{n}\left|\frac{obs_i - for_i}{obs_i}\right|}{n} \tag{13}$$

Table 1. List of relations presented for estimating fall velocity.

Originator	Main relation	Modified relation
(Zanke 1977) (EE1)	$w = \frac{10\,\upsilon}{d}\left\{\left(1+0.01\frac{(s-1)g\,d^3}{\upsilon^2}\right)^{0.5}-1\right\}$	$R_e = 2.5\left\{\left(16+0.16\,D_{gr}^3\right)^{0.5}-4\right\}$
(Zhang 1993) (EE2)	$w = \sqrt{\left[(13.95\frac{\upsilon}{d})^2+1.09(s-1)gd\right]}-13.95\frac{\upsilon}{d}$	$\mathrm{Re} = \left\{(13.95^2+1.09D_{gr}^3)^{0.5} -13.95\right]$
(Soulsby 1997) (EE3)	$w = \frac{10.36\,\upsilon}{d}\left\{(1+0.156\frac{(s-1)g\,d^3}{16\,\upsilon^2})^{0.5}-1\right\}$	$\mathrm{R}_e = 2.59\left\{\left(16+0.156\,D_{gr}^3\right)^{0.5}-4\right\}$
(Cheng 1997) (EE4)	$\frac{w\,d}{\upsilon} = \left(\sqrt{25+1.2d_*^2}-5\right)^{1.5}$ $d_* = \left(\frac{(s-1)g}{\upsilon^2}\right)_d^{0.333}$	$\mathrm{R}_e = \left\{\left(25+1.2D_{gr}^2\right)^{0.5}-5\right\}^{1.5}$
(Jimenez 2003) (EE5)	$\frac{w}{\sqrt{(s-1)g\,d}} = \frac{1}{A+\frac{B}{S_*}}$ $S_* = \frac{d}{4\upsilon}\sqrt{(s-1)g\,d}$	$\mathrm{R}_e = \frac{D_{gr}^3}{0.954D_{gr}^{1.5}+20.48}$
(Wu & Wang 2006) (EE6)	$w_s = \frac{M\nu}{Nd}\left\{\sqrt{\frac{1}{4}+\left(\frac{4N}{3M^2}D_*^3\right)^{1/n}}-\frac{1}{2}\right\}^n$	$R_e = \frac{M}{N}\left\{\sqrt{\frac{1}{4}+\left(\frac{4N}{3M^2}D_{gr}^3\right)^{1/n}}-\frac{1}{2}\right\}^n$

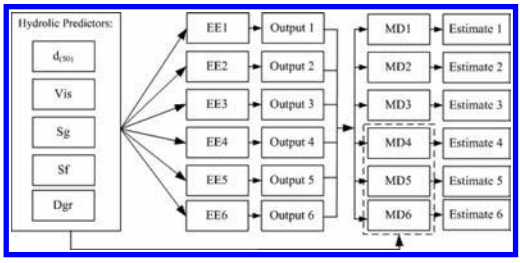

Figure 2. The schematic of the procedure used in this study.

The $\%VE$ statistic measures the absolute relative bias error of estimation values. The final statistic is the correlation ($CORR$), which measures the linear correlation between the observed and estimated data.

6 DATA SETS

Six data sets, introduced by Jimenez and Madsen (2003), are used for estimating models (Table 2). In Table 2, the number of data points from each source, n, is listed in the third column. The data sets, first, grouped into three groups. The first group, were taken from Cheng (1997), Engelund and Hansen (1972), and Hallermeir (1981), corresponds to the settling velocities of natural sediments without an explicit definition of the shape factor, but taking it as equal to 0.7, as it is usually taken as the most common value for naturally shaped sediments [for example, see Dietrich (1982)]. The Cheng (1997) data set is a compilation of Russian quartz sand experiments (original references can be found in Cheng's paper) in which the sediment size was characterized through the arithmetic average diameter (Cheng 1998). Because the specific gravity, s, was not given, it was assumed to be 2.65. The kinematic viscosity of the fluid was calculated as corresponding to fresh water at the specified temperature. The Engelund and Hansen (1972) data set was taken from Fredsoe and Deigaard (1992). The sediment size was characterized through the sieve diameter d_s (not used here), and the nominal diameter d_N and settling velocities were measured at 10°C and 20°C.

Similar to Cheng's 1998 data set, the specific gravity s was not given, so it was assumed to be 2.65 and the kinematic viscosity of the fluid was calculated corresponding to fresh water at the specified temperature. The Hallermeir (1981) data set is a compilation of previously published experiments (original refrences can be found in Hallermeier's paper), in which the sediment size was characterized by the sieve diameter.

Since all the methods here was derived to be used with the nominal diameter, the given sieve diameters were, as previously mentioned, converted to nominal diameter by using the rule of thumb $d_s/d_N \approx 0.9$ (Raudkivi 1990). As an example of the applicability of this approach, the ratio calculated from the data supplied by Engelund and Hansen (1972) gives a mean value of 0.93 with a standard deviation of 0.04. The analysis in this research is restricted to sands having size in the quartz range. Hence, only experiments with a specific gravity between 2.57 and 2.67 were considered. The kinematic viscosity was given for some experiments, and when it was not, a value of 10^{-6} m^2/s was assumed (Hallermeier 1981). The original compilation of Hallermeier's data set also included the Engelund and Hansen (1972) data, but it is considered in this research, separately.

The second group of data sets corresponds to the sediment settling velocities reported by Raudkivi (1990), originally given by the U.S. Inter-Agency Committee. The reported data consisted of settling velocities of sediment characterized by its nominal diameter and shape factor (Table 2). Finally, the total set of generated patterns has been divided into three sub-sets immethodically. About 23 patterns are kept aside for validation, 23 patterns for testing and remaining 69 were used for training the neural network.

7 THE RESULTS OF EXPERIMENTAL EQUATION AND DATA FUSION ESTIMATION MODELS

In this research, six laboratory data sets, which are extracted from 115 experiments with different hydraulic and sediment situations, are used as observed data. Among the lots of experimental relations, six of them are selected reasonably as individual estimating models (EE1 to EE6). The results of applying individual models as well as the data fusion methods are shown in Table 3 and Table 4, respectively, in cross validation analysis for testing the methods on data not used in calibration.

As shown in Table 3, EE6 has resulted in the best point estimation as it has provided estimations minimum root mean square error (RSSE), percent of volume error (%VE), and maximum linear correlation between the observed and estimated data (CORR). This means that, although EE4 and EE5 have better criteria results than EE1to EE3, EE6 outperforms the other models according to the overall estimation of whole data sets. EE6 has resulted in RSSE, VE%, and CORR, equal to 14, 11, and 88 respectively.

Data fusion methods have improved the results of individual models as demonstrated in Table 4.

Table 2. Data sets used estimation models [cited in Jimenez and Madsen 2003].

Code	Data	n	d (mm)		w (m/s)	
			Range	Mean	Range	Mean
1	Cheng (1997)[a,c]	37	0.061–4.5	1.15	0.002–0.281	0.101
2	Engelund and Hansen (1972)[c,d]	22	0.100–1.9	0.580	0.005–0.170	0.063
3	Hallermeier (1981)[b,c,d]	20	0.152–0.61	0.369	0.017–0.075	0.045
4	Raudkivi (1990)[d,e]	12	0.200–2.0	0.930	0.017–0.156	0.082
5	Raudkivi (1990)[d,f]	12	0.200–2.0	0.930	0.018–0.194	0.098
6	Raudkivi (1990)[d,g]	12	0.200–2.0	0.930	0.019–0.240	0.116

[a]Arithmetic average diameter (d_n); [b]Assuming $d_s/d_N \approx 0.9$; [c]Shape factor not given; [d]Nominal diameter (d_N); [e]Shape factor = 0.5; [f]Shape factor = 0.7; [g]Shape factor = 0.9.

Table 3. The results of experimental equations in fall velocity of sediment particles.

Models	Cross validation		
	RSSE	%VE	CORR
EE1	17.5	14	86
EE2	15.5	13.5	87
EE3	17	15	85
EE4	16	13.5	84
EE5	15.5	13	84
EE6	14	11	88

Table 4. The results of data fusion models in fall velocity of sediment particles.

Methods	Cross validation		
	RSSE	%VE	CORR
DF1	14.3	11.5	95
DF2	15.8	11.2	95
DF3	16.5	13.4	93.8
DF4	12	9.9	96.5
DF5	11.5	9.4	96
DF6	9.5	6.2	98

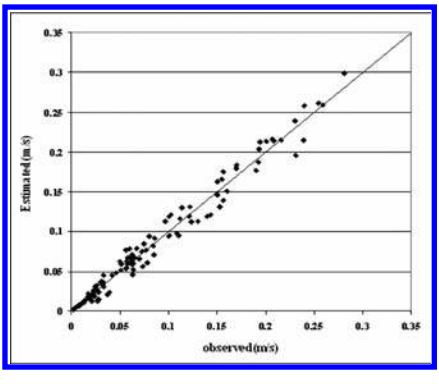

Figure 3. The results of fall velocity estimation using EE6.

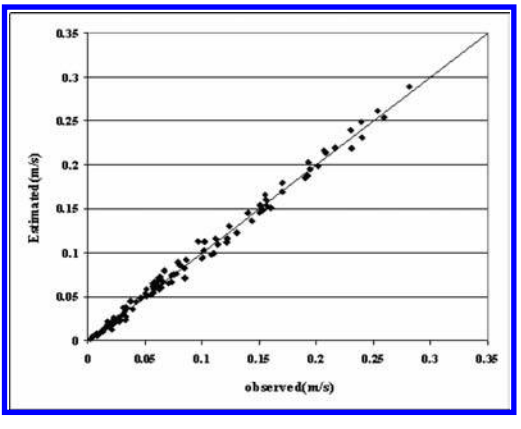

Figure 4. The results of fall velocity estimation using DF6.

The best estimation is obtained by DF6 as is has resulted in RSSE, %VE, and CORR equal to 9.5, 6.2, 98. It seems that the data fusion approach has improved the accuracy of point estimation by 4.5, 4.8, and 10 percent, according to RSSE, VE%, and CORR statistics, respectively. Methods of MD4 to MD6 have resulted in the best estimations among others. MD4 to MD6, benefit from using predictors in addition to the output of single models as well as dynamic weighting of the single models in their fusion procedure. These benefits are most likely the reasons for the supremacy of these three methods.

Among data fusion models, DF3 is obtained the worst criteria results. It could be justified due to non-linear structure of experimental equations. In fact, when output of these equations is modeled by linear regression it makes an obvious conflict between linear and non-linear structure of EE output and DF3.

Generally, in evaluating drop velocity of sediment particles, with the exception of particles diameter, other hydraulic and sediment predictors are assumed constant and just in this situation

experimental equations can estimate with desirability and out of this situation experimental equations (individual models) cannot response well, therefore ability and efficiency of data fusion methods especially DF6 will be distinguished.

8 SUMMARY AND CONCLUSION

This paper investigated the application of multi-data fusion approach into the fall velocity estimating using six methods including a new method based on the K-nearest neighbor algorithm. Six experimental equations as individual models (EE1 to EE6) were used to estimate. The improved accuracy of estimations resulted by data fusion methods, especially proposed data fusion model, confirmed that the combined estimations outperform estimations of individual models in Data sets. The results demonstrated that the proposed method outperforms other methods in Data sets.

The black-box methods (ANNs and ANFIS) were distinguished as other appropriate methods in data fusion for estimating drop velocity. As well as, it is concluded that combining methods which benefit from the use of predictors in their fusion procedure could be more promising than others. Besides, proposed method can have accurate estimations in unusual hydraulic and sediment situation while experimental equations as individual models confirm precise estimating often in usual hydraulic and sediment situation. The proposed K-NN-based method has the potential to sum up individual models in a probabilistic manner, which could be considered in future studies.

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Dam heightening

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Niveles y resguardos del recrecimiento del embalse de Yesa

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RESUMEN: El embalse de Yesa, situado sobre el río Aragón, afluente por la margen izquierda del Ebro, en las comunidades de Navarra y Aragón, tiene 447 hm^3 de capacidad y lleva en servicio desde 1959. La actual presa, de 78 m de altura sobre cimientos, es de gravedad, su paramento de aguas arriba es vertical y el de aguas abajo escalonado, con un talud equivalente a 0,78h:1v.

En la actualidad se está ejecutando el recrecimiento de dicha presa, para disponer de 1 079 hm^3 de capacidad de embalse, con una nueva presa de gravas con pantalla de hormigón de 108 m de altura, apoyada en la existente.

En la comunicación se expone una comparativa entre la situación actual con la presa de hormigón, así como el proceso seguido para la determinación del Nivel Máximo Normal del embalse recrecido. Partiendo del estudio de laminación de avenidas, junto con la identificación de los principales elementos susceptibles de ser afectados por el recrecimiento (casco urbano del pueblo de Sigüés, Camino de Santiago y ecosistemas de su entorno), se han determinado las cotas de nivel de embalse compatibles con dichos elementos, estableciéndose los correspondientes resguardos.

Con objeto de maximizar la capacidad del embalse, limitando las afecciones, y mejorar la gestión de las avenidas, el aliviadero proyectado dispone de compuertas. Sin embargo, hay que resaltar que la cota de coronación de la presa de Yesa recrecida se ha fijado con un resguardo suficiente, incluso con las compuertas del aliviadero fijas en su posición más desfavorable, para dotarla de una seguridad hidrológica adicional, ya que se trata de una presa de materiales sueltos.

Finalmente, se describe la gestión de las avenidas a realizar conjuntamente con el embalse de Itoiz, situado en el río Irati, que es afluente del río Aragón, inmediatamente aguas abajo de la presa de Yesa.

ABSTRACT: The Yesa reservoir, placed on the Aragón River, which is a tributary of the Ebro River on its left bank, in the regions of Navarra and Aragón, has a capacity of 447 Hm^3 and is in operation since 1959. The actual dam is a gravity dam with a height of 78 m. The upstream face is vertical and the downstream face is stepped with an equivalent slope of 0.78H:1V.

Nowadays, the heightening of this dam is being constructed with a new gravel-filled embankment dam, 108 m high, leant against the old dam. The aim is to increase the reservoir capacity up to 1.079 Hm^3. The impervious zone is on the upstream face and is made of concrete.

In this abstract, a comparison between the actual situation and the original concrete dam is given, as well as the assignment of the maximum normal level of the increased reservoir. Regarding the flood prevention studies, as well as the determination of the most important elements susceptible of being affected by the heightening of the dam (village of Sigües, "Camino de Santiago" pilgrimage route, ecosystems in the surroundings), the compatible water levels have been assessed, defining the freeboard.

In order to maximize the reservoir capacity, limiting the troubles and improving the flood control, the designed spillway is equipped with floodgates. Nevertheless the crest elevation of the heightened Yesa dam was determined with a sufficient freeboard, even with the floodgates fixed on its most unfavourable position, in order to give an additional hidrological security to the embankment dam.

Finally, in accordance with the Itoiz dam on the Irati River, which is a tributary of the Aragón River immediately downstream of the Yesa dam, the flood management is described.

1 ANTECEDENTES

La presa de Yesa se encuentra situada al norte de España, en el límite de las comunidades de Navarra y Aragón, dentro de la cuenca del río Ebro sobre uno de sus principales afluentes por su margen izquierda, el río Aragón.

La cuenca dominada por la presa de Yesa tiene una superficie de 2 170 km^2, con una aportación media de 1 322 hm^3/año.

El actual embalse de Yesa a su Nivel Máximo Normal alberga un volumen de 447 hm^3, y a partir de su entrada en servicio en el año 1959 ha permitido satisfacer las demandas de la zona regable de Bardenas de 85 000 ha, abastecer a los pueblos de la zona, laminar las avenidas y mantener el caudal ecológico del río Aragón.

Posteriormente a su terminación, en la década de los 70, comenzó a plantearse la necesidad de un aumento en la capacidad reguladora del embalse de Yesa. El abastecimiento de agua de calidad a 800 000 habitantes en la ciudad de Zaragoza y en su entorno, la terminación de transformación en regadío de las 110 000 ha previstas en la zona de Bardenas, así como las necesidades de agua asociadas a los cultivos actuales, motivaron la redacción en 1983 del primer proyecto de construcción del Recrecimiento del embalse de Yesa.

2 CARACTERÍSTICAS DE LA PRESA ACTUAL

La actual presa de Yesa es de gravedad de hormigón, tiene 78 m de altura sobre cimientos con

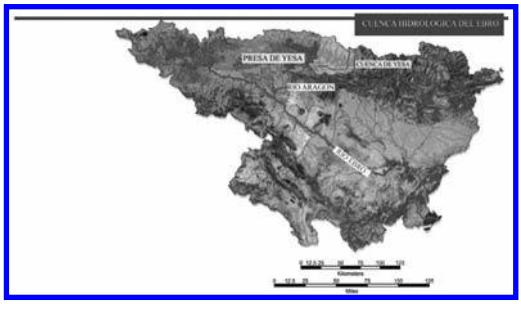

Figura 1. Situación de la presa y su cuenca vertiente.

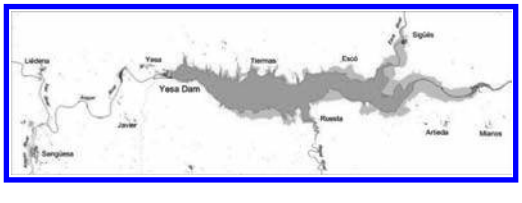

Figura 2. Planta general embalse actual y recrecido.

Figura 3. Situación de la presa antes del comienzo de las obras del recrecimiento.

un paramento vertical aguas arriba y escalonado aguas abajo con un talud equivalente de 0,78h:1v. La cota de coronación es la 490 con el Nivel Máximo Normal a la cota 488,61 msnm.

Un elemento fundamental en la determinación de niveles y resguardos en el actual embalse de Yesa es su aliviadero. Este está situado fuera de la presa y formado por cuatro pozos verticales de 6,50 m de diámetro, con compuertas de sector de 20 m de longitud de vertido y 6 m de altura.

Los pozos conectan con cuatro túneles que, tras recorrer bajo el estribo izquierdo aproximadamente 500 m, vierten a un cuenco amortiguador común a todos ellos e incorporan su caudal al río Aragón.

La presa existente se ha comportado muy satisfactoriamente durante los más de 50 años que lleva en explotación, tanto desde el punto de vista estructural como en el aspecto hidrológico, habiéndose mantenido en todo momento los usos anteriormente citados y siendo básica su aportación en la laminación de las avenidas ocurridas desde su puesta en servicio, para minimizar daños en las poblaciones aguas abajo de la misma.

3 NIVELES Y COTAS DE LA PRESA EXISTENTE

Para la realización del estudio de laminación mediante el actual embalse, es necesario asignar la avenida de proyecto correspondiente. Para ello, se ha considerado la vigente normativa española de presas que determina la obligatoriedad de clasificar las presas en tres categorías en función del riesgo potencial derivado de su posible rotura o funcionamiento incorrecto.

De este modo, la actual presa de Yesa se engloba dentro de las presas con categoría A, ya que un accidente afectaría a la población de

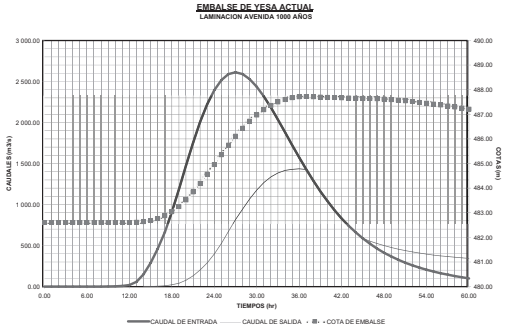

Figura 4. Laminación de avenida de proyecto mediante la presa actual.

Figura 5. Panorámica general coronación presa actual y embocadura aliviadero existente.

Sangüesa, ubicada a unos 15 km aguas abajo de la presa.

Tratándose de una presa de gravedad de hormigón con categoría A, se calcula la avenida de proyecto con un periodo de recurrencia de 1000 años, incrementándose hasta los 5000 años para el caso de la avenida extrema.

Una vez obtenidas las curvas de variación de cota en el embalse ante una situación de avenida, se han determinado los distintos resguardos a la cota de coronación de la presa.

Esta comprobación establece que la coronación de la presa de hormigón existente situada a la cota 490 m sea compatible con la laminación de las avenidas prescritas por la normativa vigente, de mayores restricciones que la empleada para la redacción del proyecto original de la actual presa, así como ante sobreelevaciones producidas por sismo o viento.

4 CARACTERÍSTICAS DE LA PRESA RECRECIDA

La zona regable del Canal de Bardenas se delimitó a comienzos del siglo XX considerándose una superficie útil de 110 000 hectáreas, suponiendo que el cultivo mayoritario consistiría en cereal de invierno (trigo y cebada) por lo que, dadas las dotaciones y la distribución temporal de los caudales demandados por estos cultivos, que son los tradicionales de la zona, permitían con el actual embalse de Yesa cubrir ampliamente sus necesidades.

La puesta en explotación de esta extensa zona regable todavía no ha finalizado, mientras que la distribución de cultivos se ha modificado con el paso del tiempo, existiendo una mayor diversificación de los cultivos (alfalfa, maíz, hortalizas, arroz, además de trigo y cebada), produciéndose unas mayores demandas de agua con una distribución a lo largo del año distinta al régimen natural del río Aragón; por este motivo, ha aumentado la dotación unitaria de agua y además se han incrementado las necesidades de regulación para adaptarse a las necesidades de los nuevos cultivos.

El recrecimiento de la presa de Yesa se realiza mediante la construcción de una presa de materiales sueltos aguas abajo de la actual, formada en su mayor parte por gravas procedentes del vaso. La nueva presa se apoya a 2/3 partes de la altura existente sobre la presa de hormigón, con una pantalla de hormigón armado de 30 cm de espesor.

El aliviadero de la presa recrecida se mantiene en la posición actual; para ello se aprovechan los cuatro túneles existentes y se recrecen sus embocaduras en los pozos de entrada. El nuevo aliviadero dispondrá de compuertas en dos de los cuatro pozos para maximizar la cota del Nivel Máximo Normal del embalse recrecido, sin llegar a afectar a ninguna población de su vaso. Además, de este modo, se consigue una mejor laminación de avenidas al actuar conjuntamente con el embalse de Itoiz, cuyo aliviadero tiene labio fijo.

Si se hubiera optado por disponer de compuertas en los cuatro vanos del aliviadero, la altura de las compuertas debería haber sido menor, para que los daños por un vertido incorrecto no se incrementaran. Sin embargo, el volumen de embalse dominado sería menor, lo cual no es conveniente puesto que las compuertas del aliviadero tienen función semejante a la de una toma superior.

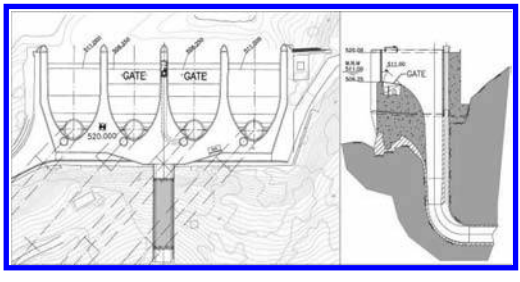

Figura 6. Aliviadero presa recrecida. Planta y sección.

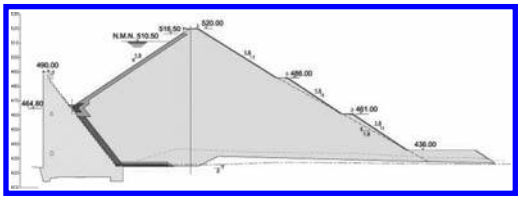

Figura 7. Sección tipo Recrecimiento presa de Yesa.

Se ha optado por disponer dos compuertas en lugar de una para facilitar las labores de mantenimiento y reducir las limitaciones de llenado en caso de que una compuerta se encuentre fuera de servicio.

Aunque las compuertas del aliviadero son necesarias por los dos motivos expuestos, la cota de coronación de la presa recrecida se ha establecido sin considerar su funcionamiento.

Con esta solución, se incrementa 30 m la altura sobre cimientos de la presa actual y se aumenta la superficie de embalse de 2 233 ha a 3 584 ha, alcanzándose un volumen superior al doble del embalse actual pasando de 447 hm^3 a 1 079 hm^3.

4.1 *Umbrales afección Recrecimiento*

En cuanto a las afecciones producidas por la nueva presa, cabe destacar que éstas no se producen de manera progresiva sobre los distintos elementos, sino que van asociadas a unos umbrales de afección a distintas cotas, las cuales son esenciales para determinar las dimensiones de la presa recrecida.

5 SIGÜÉS

La población de Sigüés se asienta sobre el río Esca, afluente del río Aragón por su margen derecha. El centro de esta localidad se encuentra a la cota 519,00 m mientras que la última calle del casco urbano se sitúa a 512,50 m.

Para compatibilizar el mantenimiento del pueblo evitando la creación de una franja afectada por la carrera del embalse, se proyecta un dique con amplitud suficiente entorno al casco urbano, favoreciendo la creación de una zona de servicios, instalaciones deportivas y jardines.

Para la corrección total del impacto sobre el núcleo urbano de Sigüés, se requiere que el nivel del agua del embalse alcance siempre el pie de dicho muro, para lo que se proyecta una presa de cola en el río Esca. Esta presa estará dentro del embalse recrecido, sin crear por lo tanto nuevas afecciones. La presa de cola dispondrá de un desagüe de fondo que permitirá su vaciado, posibilitando la utilización del volumen embalsado (3 hm^3), en caso de necesidad, por lo que su construcción no supone reducción del volumen útil del embalse de Yesa.

El empleo de las compuertas previstas en el nuevo aliviadero, permitirá reducir la diferencia entre el Nivel Máximo Normal del embalse y la Cota de Coronación del dique de protección del casco urbano de Sigüés, maximizando el volumen de embalse.

6 CAMINO DE SANTIAGO

Asociado al Camino de Santiago, junto al embalse de Yesa, se sitúa a la cota 515,00 m la ermita de San Juan en Ruesta, románica del siglo X.

Limitando el recrecimiento a esta cota, se evitan las afecciones a sus principales elementos asociados y únicamente se precisa la reposición de 4,2 km del actual trazado del Camino para mantener el paso de los peregrinos y reducir las afecciones sobre esta vía.

7 ECOSISTEMAS

Con el objetivo de no superar un impacto bajo sobre los ecosistemas más interesantes y zonas de especial protección, se ha establecido no superar la cota 512,00 m como Nivel Máximo Normal en el recrecimiento del futuro embalse.

8 NIVELES Y COTAS PARA LA PRESA RECRECIDA

8.1 *Estacionalidad de avenidas*

Durante el verano, el embalse de Yesa, coincidiendo con las menores aportaciones del río Aragón, debe suministrar los mayores volúmenes de agua por él regulados, destinados principalmente al riego de los cultivos agrícolas. Esta situación hace que la explotación del embalse procure tenerlo lo más lleno posible al inicio del verano para finalizar este trimestre con el menor volumen de todo el año.

Las mayores avenidas se presentan en el embalse de Yesa en otoño y las siguientes en primavera, calculándose los resguardos necesarios para esta época, en la que el embalse debe estar lo más lleno posible antes de comenzar la campaña de riego. De esta forma, se fija el máximo nivel de embalse en primavera, que es válido tanto para invierno como para verano, ya que sus avenidas son menores. El máximo nivel de embalse en otoño se determina reduciéndose el establecido para primavera, de forma que pueda laminarse sus mayores avenidas disponiendo de mayores resguardos, sin que ello afecte la capacidad de regulación del embalse, puesto que al principio del otoño finaliza la campaña de riego.

8.2 *Avenida de diseño*

Para fijar la avenida de proyecto y la avenida extrema a contemplar para la determinación de cotas y resguardos de la presa recrecida, es necesaria la clasificación de ésta y del dique de Sigüés, considerando a este último como una presa de hormigón.

De manera evidente, al tratarse de un recrecimiento de una presa con categoría A, la nueva presa de Yesa mantiene esta clasificación. Así, como avenida de proyecto para la presa recrecida se considera un periodo de recurrencia de 1000 años mientras que para la avenida extrema, se establece un periodo de 10 000 años al tratarse de una gran presa de materiales sueltos con categoría A.

Sin embargo, en el caso del dique de protección de Sigüés, la máxima inundación que ocasionaría su potencial rotura alcanzaría la coronación del muro, produciéndose en todo caso una inundación remontante hacia el casco urbano situado a media ladera, no produciéndose daños materiales ni medioambientales importantes.

En consecuencia, al muro de protección de Sigüés, realizando su clasificación como si se tratase de una presa de hormigón, se le asigna la categoría B. Siguiendo lo estipulado para presas con categoría B, se dispone como avenida de proyecto la de 500 años de periodo de retorno y 1000 años en el cálculo de la avenida extrema.

Adicionalmente, como consecuencia de la forma del embalse de Yesa, para el cálculo de la formación de oleaje producido por el viento, resulta que la máxima distancia de la superficie de agua (fetch) a la cota del embalse recrecido en la presa es de 20 km, mientras esta distancia máxima en dirección al casco urbano de Sigüés es de 5 km. Estos datos implican alturas de ola diferentes para estos puntos y, por lo tanto, requieren distintos resguardos.

Una vez determinados estos datos de partida, el Nivel de Avenida de Proyecto (NAP) y el Nivel de Avenida Extrema (NAE) para la futura presa recrecida y para el dique de protección de Sigüés se han calculado laminando la avenida de periodo de retorno correspondiente, suponiendo el embalse a su Máximo Nivel Normal. (Fig. 8)

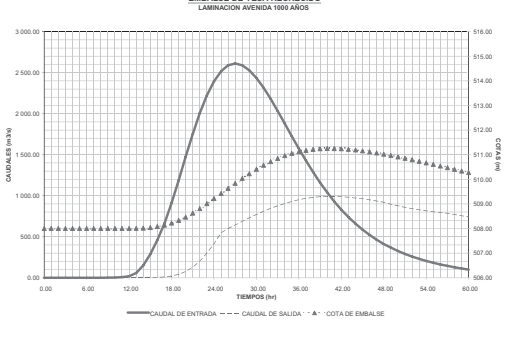

Figura 8. Laminación de avenida de proyecto mediante la presa recrecida.

8.3 *Comprobación de resguardos para la presa recrecida*

La capacidad de laminación ante una avenida es significativamente mayor para el caso del embalse recrecido en comparación con la realizada por el embalse actual. Este hecho implica una seguridad hidrológica adicional ante posibles daños aguas abajo de la presa como consecuencia de la evacuación de caudales punta de menor magnitud.

Adicionalmente a los niveles de avenida, también se considera la amplitud de la onda del oleaje producida por el viento. Este valor se obtiene en función de la máxima longitud de fetch aplicando la fórmula de Stevenson ($A = 0.76 + 0.34 \cdot F^{1/2} - 0.26 \cdot F^{1/4}$). La sobreelevación debida a la ola máxima producida por el viento (SOM), se obtiene incrementando la anterior en un 50% (SOM = 1,5·A). Como sobreelevación debida a la ola máxima producida

Comprobacion cota de coronacion de la presa recrecida y muro de sigües

		Presa recrecida	Muro sigües
Cota de coronación	**m**	**520**	**514,25**
Nivel Máximo Normal (NMN)	m	511	511
Periodo de retorno Avenida de Proyecto (Tr NAP)	años	1000	500
Nivel de Avenida de Proyecto (NAP)	m	**512,71**	**512,41**
Periodo de retorno Avenida Extrema (Tr NAE)	años	10 000	1000
Nivel de Avenida de Extrema (NAE)	M	**513,94**	**512,71**
Longitud de fetch	km	20	5
Ola máxima producida por el viento	m	2,6	1,7
Ola máxima producida por el viento durante una avenida	m	1,82	1,19
Ola máxima producida por sismo	m	1,89	1,37

Figura 9. Comprobación de resguardos a coronación de la presa recrecida y al muro de protección de Sigüés.

por el viento en avenidas (SOA), se considera el 70% de esta última (SOA = 0,7 SOM).

La altura de ola por sismo (AS) se calcula, para cada emplazamiento, en función de las características de la presa, principalmente su altura máxima a cimentación. A efectos de resguardo, se considera como sobreelevación máxima debida a la ola sísmica (SOS), el valor de la altura de ola por sismo incrementado en un metro (SOS = AS + 1 m).

Asimismo, el máximo nivel alcanzado durante la avenida de proyecto para el embalse recrecido se corresponde con la cota 512,71 m, aumentando hasta 513,94 m para la situación de avenida extrema.

Teniendo en cuenta los distintos periodos de recurrencia para el caso del cálculo de avenida en el muro de Sigüés, así como los establecimientos de resguardos por oleaje sensiblemente menores por los motivos anteriormente descritos, los resultados obtenidos indican un nivel máximo alcanzado en régimen de avenida extrema en este punto a la cota 512,71 m.

De esta manera, se ha determinado la cota 514,25 m como nivel de coronación del dique de protección de Sigüés, maximizando el volumen del embalse a recrecer siendo compatible en todo momento con el mantenimiento de la población.

Para el caso de la futura presa, el establecimiento de la cota de coronación a la cota 520 m permite laminar las avenidas prescritas por la normativa en vigor incluso con las compuertas fijas en su posición más desfavorable, incrementando sustancialmente la protección ante avenidas respecto a la situación actual de la propia presa y de las poblaciones aguas abajo de la misma.

Los cálculos realizados determinan que la diferencia del nivel alcanzado en el embalse al dejar de funcionar una compuerta, manteniéndose las restantes hipótesis de laminación, es inferior a 1 m; por lo que, se reducirá el nivel estacional del embalse en un metro por cada compuerta del aliviadero que esté fuera de servicio.

9 LA SEGURIDAD HIDROLÓGICA DEL RECRECIMINENTO DE LA PRESA DE YESA

Entre los usos principales derivados del recrecimiento, como son el abastecimiento a Zaragoza y su entorno junto con el regadío de 110,000 ha en la zona de Bardenas, se consigue un aumento de la capacidad reguladora del embalse, lo que supone una seguridad hidrológica adicional respecto al actual.

Los niveles establecidos para la presa actual, así como los proyectados en el caso del recrecimiento, permiten en todo caso la gestión de las avenidas de proyecto y extrema respetando en todo momento los resguardos mínimos a la coronación de la presa.

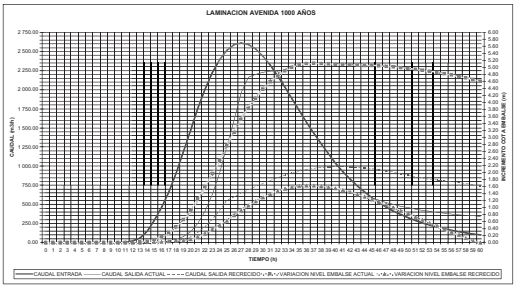

Figura 10. Comparativa laminación avenida proyecto por la presa actual y por la presa recrecida.

Cabe destacar la significativa mejora en la capacidad laminadora del embalse recrecido, ya que ante una misma situación de avenida, el incremento de nivel de embalse es menor, reduciéndose por lo tanto el caudal máximo de vertido durante este proceso.

10 REGULACIÓN DEL RÍO ARAGÓN MEDIANTE LOS EMBALSES DE YESA E ITOIZ

Los ríos Aragón e Irati confluyen aguas arriba de la localidad de Sangüesa. Sus cuencas vertientes poseen una superficie semejante, por lo que cuando se producen periodos intensos de lluvia, los caudales máximos de ambos ríos suelen coincidir en el tiempo, produciéndose un caudal punta conjunto en la población de Sangüesa.

Esta población Navarra ha sufrido históricamente inundaciones de importancia, situación que también se ha dado en todos los municipios navarros situados a orillas del río Aragón. Entre los episodios de avenidas históricas, hay que destacar las inundaciones producidas en Sangüesa el 23 de mayo de 1582, que inundo más de la mitad de la villa, o la que tuvo lugar en septiembre de 1787, donde perecieron 585 personas a consecuencia de las inundaciones.

Desde el año 1959, se encuentra en explotación el embalse de Yesa, que se ubica justo aguas arriba de la confluencia con el río Irati, situándose seguidamente de dicha confluencia la población de Sangüesa. La existencia de este embalse ha supuesto una reducción importante de los caudales máximos de avenidas y, dado que las principales se producen en otoño cuando el embalse no está a su máximo nivel, se reduce sustancialmente el caudal saliente de la presa. Es el caso de las avenidas producidas en Noviembre de 1984 y Diciembre de 1996, cuyos caudales punta de entrada al embalse superaron los 900 m^3/s mientras que el caudal de salida del mismo quedo reducido a 8 m^3/s.

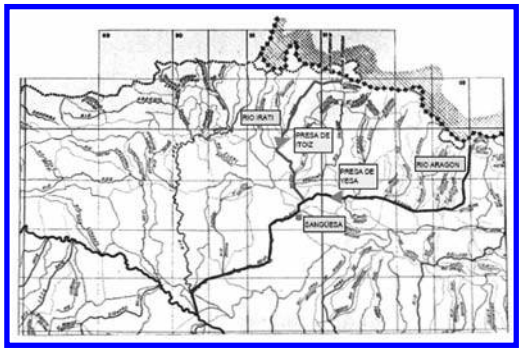

Figura 11. Planta general cuencas vertientes ríos Irati y Aragón en el entorno de Sangüesa.

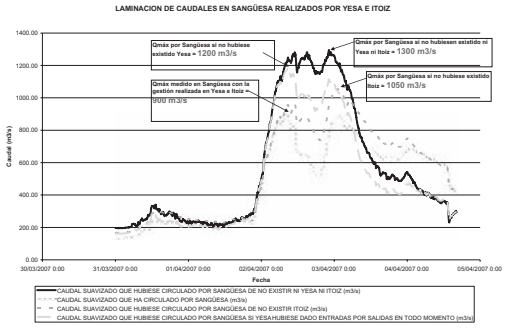

Figura 12. Laminación de caudales en Sangüesa realizados por Yesa e Itoiz en 2007.

Cualquier embalse, aun encontrándose lleno, permite conseguir que el caudal de salida sea menor al de entrada ya que una parte de este último debe emplearse en ocupar el resguardo previsto para las avenidas.

Tanto en la actual presa como en el recrecimiento de la presa de Yesa, se disponen compuertas en el aliviadero, lo que permite vaciar parcialmente el embalse cuando se detectan fuertes lluvias en su cuenca con anticipación a la llegada de esta agua al embalse. Así, se consigue vaciar parcialmente el embalse, disponiéndose de un volumen adicional para retener parte de la avenida, reduciéndose sustancialmente el caudal de salida.

En la situación actual, aguas arriba de Sangüesa, existe el embalse de Yesa que cierra la mayor parte de la cuenca del río Aragón, aguas arriba de su confluencia con el Irati, y el embalse de Itoiz, que se sitúa en el propio río Irati existiendo parte de sus afluentes sin regular, entre los que destacan los ríos Salazar y Erro.

Con estas condiciones, la gestión de las avenidas en Yesa tiene en cuenta por una parte la previsión de precipitaciones y fusión de nieve en la cuenca, y por otra los datos de caudales circulantes en los cauces. En base a ello, se gestiona la apertura y cierre de las compuertas de forma que los caudales que se sueltan al río no coincidan con los máximos caudales naturales de éstos, evitando que se sumen las puntas de las avenidas.

En la Figura 12, se ha representado la avenida real producida en Abril de 2007. La adecuada gestión del actual embalse de Yesa, utilizando la información recogida en el S.A.I.H. (Sistema Automatizado de Información Hidrológica) junto con la laminación producida en la presa de Itoiz, permitió limitar el caudal circulante en Sangüesa a 900 m^3/s, evitando daños en esta población y los restantes municipios situados en las proximidades del río Aragón. Las avenidas que se hubieran presentado en régimen natural en Sangüesa hubieran alcanzado los 1 300 m^3/s, produciendo graves afecciones en este municipio,

Figura 13. Recrecimiento de la presa de Yesa.

así como en los situados aguas abajo de él, daños que se evitaron con la adecuada gestión de estos embalses.

11 CONCLUSIONES

Los cálculos realizados, partiendo del Nivel Máximo Normal del embalse de Yesa recrecido, considerando los niveles y cotas asignados a los distintos elementos del entorno del vaso, demuestran su capacidad para laminar las avenidas, evitándose el traslado tanto del casco urbano de Sigüés (cota 514,25 m), como de las ermitas del Camino de Santiago, y generan un impacto bajo sobre los ecosistemas de su entorno. Objetivos que se logran garantizando la seguridad hidrológica del recrecimiento de la presa de Yesa y maximizando el volumen regulado por el embalse recrecido.

Finalmente, hay que resaltar que la coronación de la presa de Yesa recrecida se ha situado a cota 520,00 m, para dotarla de una seguridad hidrológica adicional: que la proteja incluso con las compuertas del aliviadero enclavadas en su posición más desfavorable, con el nivel normal de embalse más alto posible y sin considerar reducción de la altura de ola de viento en avenidas.

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Recrecimiento de la presa de Las Torcas

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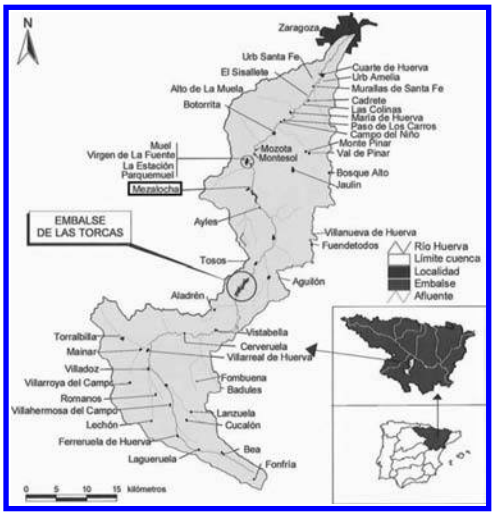
RESUMEN: La presa de Las Torcas, en servicio desde 1948, es la pieza fundamental del sistema de regulación del río Huerva, que cuenta además con la presa de Mezalocha, aguas abajo. En su estado actual tiene un perfil de gravedad más bien estricto y un aliviadero de escasa capacidad. Además se ha constatado la falta de garantía de suministro a los riegos de la cuenca media y baja del río, por lo que se plantea la necesidad de su recrecimiento. Aunque no se llegara a aumentar el volumen de explotación, deberían arbitrarse medidas para aumentar la seguridad de la presa (incrementando la capacidad de vertido por el aliviadero, disponiendo de galerías de drenaje e inspección, ...). Como consecuencia de los estudios realizados, se ha seleccionado como más idónea una solución de recrecimiento por aguas abajo, en una altura de 14,30 m, resultando una altura total de 52 m.

ABSTRACT: The Las Torcas Dam, in service since 1948, is the cornerstone of the regulatory system of the Huerva river, which also has the Mezalocha dam downstream. In its current state it has rather strict profile of gravity and a limited capacity spillway. It is also noted that there is not a guarantee of water supply for irrigation of the middle and lower river basin, so arises the need for its regrowth. Even if it is not necessary to increase the volume of exploitation, messures should be put in place to improve dam safety (increasing the discharge capacity of spillway, having drainage and inspection galleries, …). As a result of researching the issue, it has been decided that the most suitable solution for regrowth downstream is an additional height of 14,30 m, resulting in a total height of 52 m.

1 INTRODUCCIÓN. NECESIDAD DE REGULACIÓN DE LA CUENCA

En la cuenca del Ebro, las aportaciones de la margen izquierda son mucho mayores que las de la margen derecha. La causa de esta diferencia es la debilidad con que los frentes de lluvia atlánticos llegan a las cuencas de los ríos Queiles, Jalón, Huerva, Aguas Vivas, Martín, Guadalope y Matarraña. Las lluvias procedentes del Mediterráneo son de carácter torrencial, lo que con cierta frecuencia provoca la destrucción de las cosechas y de las obras existentes en los cauces. Esta escasez de caudales (salvo en raras y torrenciales ocasiones) tiene como consecuencia que los regantes y demás usuarios de aprovechamientos de agua estén mentalizados en la necesidad de regulación de las cuencas desde hace mucho tiempo.

Así la presa de Mezalocha, con 3 hm^3 de embalse, se construyó entre los años 1896 y 1903 sobre el emplazamiento de otra más antigua que se arruinó en 1766 y la presa de Las Torcas, proyectada en 1922, con 6,7 hm^3 de embalse, se terminó de construir en el año 1948.



2 LA PRESA DE LAS TORCAS EXISTENTE

La presa del embalse de Las Torcas se encuentra situada sobre el río Huerva, afluente por la margen derecha del Ebro, y a unos 2,7 km aguas arriba de la población de Tosos dentro de su término municipal, en la provincia de Zaragoza, a una distancia de 20 km de Cariñena y 68 km de Zaragoza.

El río Huerva desemboca en el Ebro, en la misma capital, a los 143 km de recorrido y después de pasar bajo el Canal Imperial.

La cuenca vertiente al embalse, de 456 km^2, se desarrolla entre la cota 1.300 en el nacimiento del río en la Sierra de Cucalón, en Fonfría, provincia de Teruel y la cota 600.

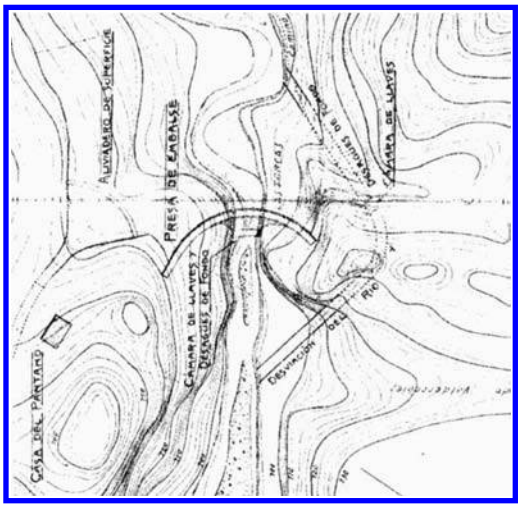
El proyecto primero de la presa data de 1922, si bien la solución inicialmente planteada en arco fue sustituida por la tipología definitiva en el proyecto reformado de 1935.

La presa, situada en el portillo de Las Torcas de Tosos, -aquí la palabra Torca tiene una acepción de meandro-, con una capacidad actual de embalse de 6,66 hm^3, se inició en diciembre de 1940 y fue terminada en 1948, estando formada por un dique de gravedad de planta recta con una altura sobre el cauce de 32,50 m.

La longitud de coronación es de 100,50 m y el ancho de la misma es de 4,60 m.

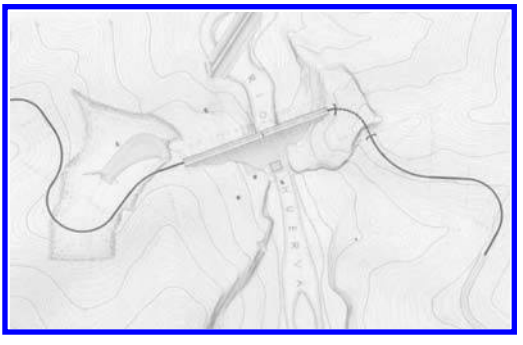
La sección tipo de la presa es triangular con talud aguas arriba vertical y de 0,77 aguas abajo. La coronación está constituida por un triángulo análogo al de la presa pero invertido, produciéndose el acuerdo entre ambos triángulos mediante un arco de circunferencia.

En la margen derecha de la presa está ubicada la antigua galería de desvío del río, con un tapón de

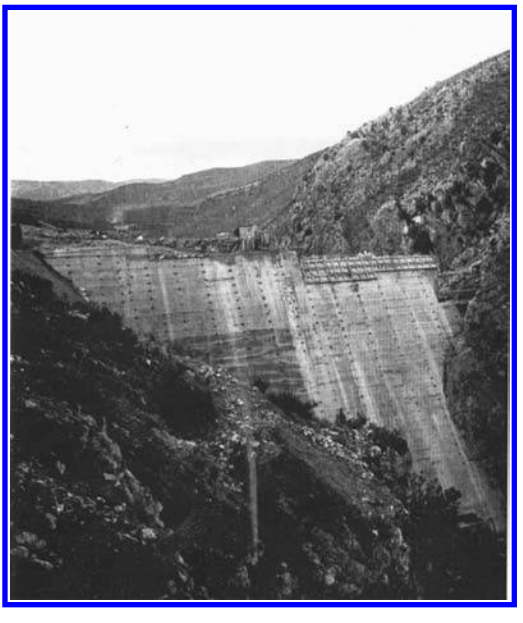


Planta general de las obras según proyecto de 1922.

Portillo de las Torcas, fotografía tomada en 1926.



Planta general de las obras según proyecto de 1940.



Vista del paramento aguas abajo durante la construcción.

hormigón que presenta una abundante filtración. En dicha margen e independiente del cuerpo de presa, se dispone también el aliviadero, inicialmente de labio fijo, en el que se han instalado en 1972 dos compuertas tipo vagón, que elevan la cota máxima del embalse hasta su vértice teórico. Los caudales vertidos se desaguan aguas abajo de la presa a través de un túnel.

Para el Nivel Máximo Normal del embalse (624,35 msnm) la capacidad de descarga del aliviadero de compuertas, con una altura de lámina vertiente de 5 metros, es de 190 m^3/s.

No dispone de ninguna galería de inspección y drenaje en el cuerpo de presa. En el centro de la presa y adosado al talud de aguas abajo se sitúa la cámara de llaves de los desagües de fondo, formados por dos conductos de 1,000 mm de diámetro.

En relación con el hormigón de la presa actual hay que señalar que se encuentra en buen estado. De las probetas extraídas se deduce una densidad aparente media de 24,4 kN/m^3 (2,44 Tn/m^3) con una porosidad del 10% (como comparación el hormigón de la presa de Mezalocha, con una altura sobre cimientos de 36 m y ejecutada a principios del siglo XX, tiene una densidad aparente media de 19,2 kN/m^3 (1,92 Tn/m^3) con una porosidad del 28%—CEDEX, 2004-).

Aliviadero de labio fijo y compuertas aguas abajo.

Vista general del paramento de aguas abajo y caseta de válvulas.

De acuerdo con los resultados obtenidos en los ensayos de resistencia realizados se observa una resistencia a la compresión simple promedio de 19,8 N/mm^2 (198 Kp/cm^2).

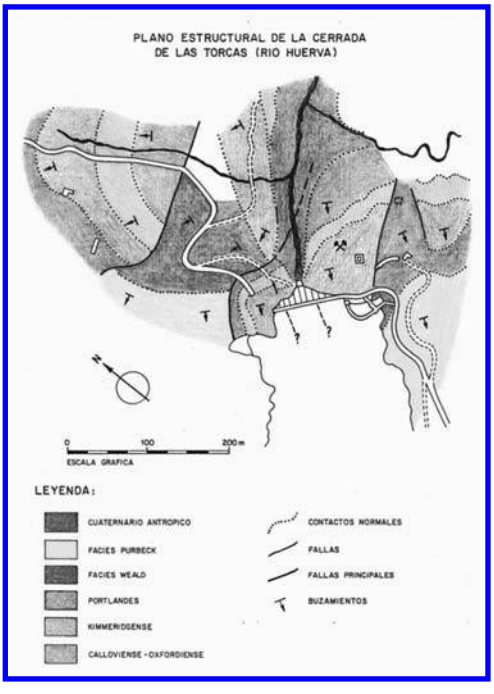
Respecto al Módulo de Young, el valor promedio de los ensayos realizados es del orden de 24.457,94 N/mm^2 (244.579,40 Kg/cm^2).

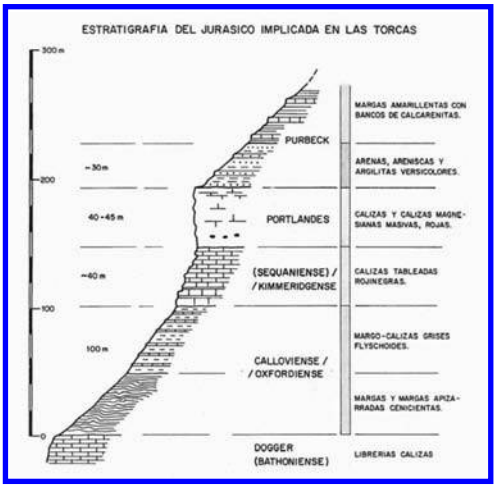
En relación con la cimentación hay que señalar que la presa se apoya en el flanco izquierdo de un anticlinal sobre calizas jurásicas compactas algo estratificadas (margocalizas en la formación Calloviense, calizas tableadas en la formación Kimmeridgense y masivas en la Portlandiense), con buzamiento favorable a la estabilidad de la presa.

Como consecuencia de la formación del anticlinal se aprecian en el terreno fallas, corrimientos en masa, resbalamientos y que las calizas se encuentren frecuentemente milonitizadas, produciendo el efecto de una geología muy complicada cuando la causa fue relativamente sencilla. En el "Estudio de las condiciones geológicas del emplazamiento y del vaso del Pantano de Las Torcas en el río Huerva" realizado por el Ingeniero de Caminos, Canales y Puertos Clemente Sáenz en el año 1926, antecedente que hemos tenido el privilegio de analizar se hace un estudio detenido de las fallas y de "las dislocaciones que presenta el terreno".

Se publicó dicho estudio en 1929 como Monografía por la CHE, prologada por M. Lorenzo Pardo. De dicho prólogo se extrae el siguiente párrafo "Sáenz se queja de que la rapidez con que inspeccionó el país sólo le permitiera recoger y poseer un ejemplar que atribuye al poly plocus Perispinates; pero nosotros que le hemos visto en casos semejantes cubrir en sus recorridos anchuras amplísimas haciendo uso del martillo, del bastón, de la navaja y hasta de las uñas propias; que hemos comprobado que los fósiles le acuden a la mano, como las buenas piezas a la escopeta del cazador de raza, no dudamos de tal rareza; aún añadiríamos que estamos dispuestos a asegurar que se trata de un ejemplar aislado, porque siempre hemos creído que Sáenz no encuentra casualmente los fósiles, sino que los tiene citados, dándose siempre, o casi siempre, el caso de acudir puntualmente los del correspondiente y anunciado tramo."

Si bien el proyecto analizado por Clemente Sáenz contemplaba una solución en arco delgado, que consideraba en dicho estudio compatible con las condiciones topográficas y geotécnicas del emplazamiento, el asesor del entonces Consejo Técnico de Construcción, D. Agustín Marín,





propuso su modificación a presa de gravedad de planta recta "por las dislocaciones que presenta el terreno", que fue recogida en el proyecto reformado de 1935.

3 NECESIDAD Y PROPÓSITO DE LA AMPLIACIÓN DEL EMBALSE DE LAS TORCAS

El comportamiento irregular del río Huerva se observa claramente en la serie de aportaciones al embalse de Las Torcas, con un valor medio anual próximo a 30 hm^3/año, se han registrado valores próximos a 100 hm^3/año y mínimos del orden de 6 hm^3/año.

El embalse pasa sucesivamente, año tras año, de estar lleno y vertiendo a quedarse vacío.

Los estudios realizados ponen en evidencia la falta de garantía de suministro aceptables para los usos actuales y futuros previstos para la cuenca del Huerva en el Plan Hidrológico de la Cuenca del Ebro, y, en consecuencia, la necesidad de proyectar su recrecimiento. Además, la reciente sucesión de años secos ha tenido gran repercusión en los regadíos, por lo que los usuarios a través del Sindicato Central del río Huerva han solicitado que se acometieran lo más urgentemente posible dichas obras de recrecimiento.

En el Plan Hidrológico del Ebro se recoge la ejecución, en el primer horizonte del mismo, de las obras de recrecimiento de la presa de Las Torcas, para conseguir una capacidad de almacenamiento de unos 14 hm^3, requerida para garantizar las demandas de abastecimiento, regadío e industrial del sistema

Vista general del embalse.

Embalse desde aguas arriba.

Embalse desde aguas abajo.

del río Huerva. Dicha capacidad de almacenamiento se consigue situando el nuevo máximo nivel normal del embalse 8,65 m más alto que el actual.

Los beneficios adicionales del aumento del embalse son:

– Mayor capacidad de laminación de las avenidas del río Huerva, de carácter torrencial, que redunda en todas las poblaciones situadas aguas abajo, incluida Zaragoza, amortiguando los caudales punta, retrasándolos y reduciendo los daños aguas abajo.
– Creación de un embalse de nivel constante en una de las colas del embalse, con objeto de favorecer el establecimiento de la fauna.

4 JUSTIFICACIÓN DE LA SOLUCIÓN ADOPTADA

España, al igual que otros países que por la irregularidad de sus ríos tuvieron que recurrir desde antaño a la creación de embalses, ha necesitado llevar a cabo obras de recrecimiento de los mismos para poder hacer frente a demandas de agua crecientes.

El recrecimiento de presas no es pues en España una novedad. Ya en el siglo II de nuestra era se recreció 15 metros de altura la presa de Proserpina, uno de los principales monumentos de la ingeniería hidráulica romana en Hispania, construida en su primera fase en el siglo I a.c.

Mediante el recrecimiento de la presa se obtiene una nueva estructura de mayores dimensiones capaz de resistir las nuevas cargas a las que estará sometida, de la misma forma que lo hubiera hecho si se hubiese construido originalmente en su dimensión final.

Centenares de presas se han recrecido en todo el mundo, una treintena de ellas en España. Las presas de tipología de gravedad -a la que corresponde la Presa de Las Torcas- han sido las más frecuentemente recrecidas.

Las diferentes soluciones de recrecimiento están condicionadas, principalmente, por los siguientes factores:

– Tipología actual de la presa: gravedad de hormigón en masa, con una longitud de coronación de 100 m y una altura desde cimientos de 37 m.
– Topografía: la cerrada aguas abajo desaparece bruscamente en la margen izquierda.
– Características geológico-geotécnicas de la cimentación: Sustrato rocoso competente.
– Altura de recrecimiento de la presa: 14,30 m.
– Necesidad de mantener en servicio un volumen mínimo de embalse durante las obras.
– La presa de Las Torcas, en su estado actual, tiene un perfil más bien estricto y un aliviadero de escasa capacidad. Aunque no se llegara a aumentar el volumen de explotación, deberían arbitrarse medidas para aumentar la seguridad de la presa (por ejemplo galería y drenajes, incremento de la capacidad de vertido por el aliviadero, ...).

Los procedimientos más comunes para llevar a cabo el recrecimiento son:

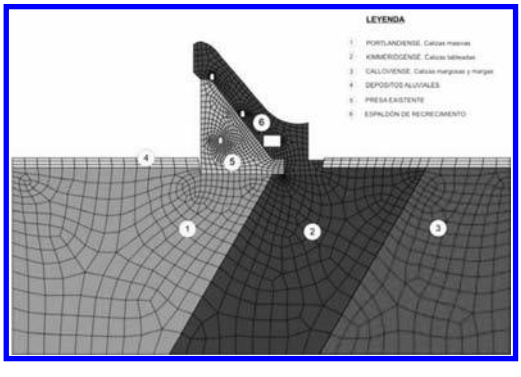
– Añadiendo masa en coronación. Es el procedimiento más sencillo por no afectar a los paramentos y no precisar la bajada del embalse. Es adecuado para recrecimientos de pequeña altura (hasta 4–5 metros). No se ha considerado la posibilidad de emplear cables postensados por falta de seguridad en el tiempo por relajamiento y corrosión La altura de recrecimiento prevista está, por tanto, fuera del límite sancionado por la experiencia, por lo que se desecha por inviabilidad técnica.
– La ejecución de una nueva presa de gravedad aguas abajo de la actual e independiente de ésta. Se ha procedido a su anteproyecto y valoración.
– Recrecimiento por aguas arriba. Requiere vaciar completamente el embalse y realizar excavaciones junto al pie de aguas arriba, fuertemente solicitado a embalse vacío. Es adecuado en presas con paramento de aguas arriba deteriorado y alturas de recrecimiento hasta 8–10 m. Esta alternativa, al exigir dejar totalmente fuera de servicio el embalse durante las obras es inviable técnicamente y se desecha.
– Recrecimiento por aguas abajo. Requiere excavar la nueva cimentación por bataches y limitar el máximo nivel de embalse durante las obras. Es el procedimiento aconsejable para recrecimientos de cierta entidad. No tiene límite de altura de recrecimiento, salvo el que establezca la calidad de la cimentación para la presa en su

dimensión final. Dentro de este procedimiento, cabe plantear dos alternativas tipológicamente diferenciadas:

- Recrecimiento con un espaldón de escollera (tipología mixta). Este procedimiento exige conectar la presa antigua a la pantalla de hormigón armado que se requeriría situar sobre el paramento de aguas arriba del espaldón de escollera. La diferente deformabilidad en el apoyo de la pantalla, por una parte en la presa existente y por otra en los estribos de las márgenes sobre el terreno natural, podría provocar asientos diferenciales con la consiguiente rotura de la pantalla. Además esta tipología mixta, al impedir verter sobre el paramento de escollera, precisa la ejecución de un aliviadero lateral para un caudal de avenida de proyecto de 633 m^3/s (y extrema de 859 m^3/s) y la adaptación del actual aliviadero en túnel para alojar un desagüe intermedio.
- Recrecimiento con un espaldón de hormigón. Este procedimiento, adoptado en España en las presas de Bolarque, Irabia y Torre de Abraham, es el elegido para el recrecimiento de la presa de Las Torcas. Es más ventajoso económicamente, presenta menos incertidumbres en cuanto a su comportamiento estructural y tiene menor incidencia paisajística y medioambiental. La altura prevista de recrecimiento es de unos 14 metros, existiendo experiencia de recrecimientos muy superiores.

De acuerdo con el Dr. B. Gilg (Suiza)—Ponente General en el Congreso Internacional de Grandes Presas (Durban, 1994)—"Es aconsejable, siempre que sea posible, recrecer por el paramento de aguas abajo, que evita el vaciado total del embalse y las excavaciones al pie del paramento de aguas arriba".

Se ha elegido, por tanto, para el recrecimiento de la presa de Las Torcas una solución convencional, contrastada, de la que se tiene multitud de ejemplos en todo el mundo, con objeto de reducir los imprevistos y eliminar los riesgos.



5 DESCRIPCIÓN DE LAS OBRAS

5.1 *Cuerpo de presa*

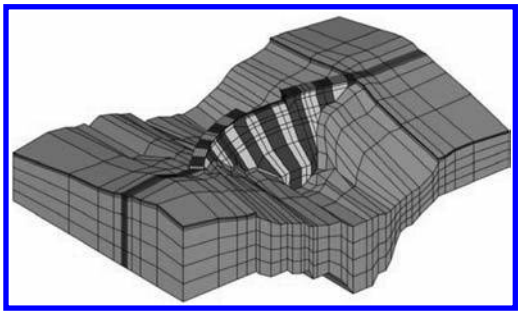
Se ha proyectado el recrecimiento de la presa existente en una altura de 14,30 m, incrementando la sección de hormigón por el paramento de aguas abajo. La nueva sección tipo es la correspondiente a una presa de gravedad de aproximadamente 52,0 metros de altura total sobre cimientos, con el paramento de aguas arriba vertical y talud 0,84H/1V en el paramento de aguas abajo. Para el recrecimiento de la sección será preciso demoler la presa existente hasta alcanzar las formas y las cotas que se señalan en los Planos.

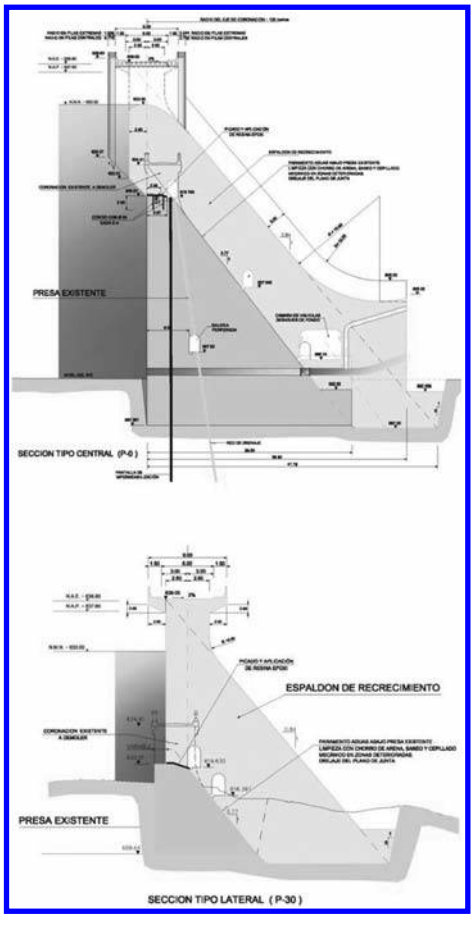
El vértice triangular de la presa se sitúa a la cota 639,00 m, por encima del máximo nivel normal del embalse. El talud del paramento de aguas abajo es 0,84H/1V, y vertical el de aguas arriba. En planta se define con dos centros, si bien la curvatura tiene por único objeto el encaje en la topografía ya que no se ha previsto la inyección de los bloques y por consiguiente el funcionamiento como arco.

El NMM (Nivel Máximo Normal) se encuentra a la cota 633,00 m y el NAP (Nivel de Avenida de Proyecto) a la 637,60 m. La longitud de la coronación (situada a la 639,00 m) es de 180 m y tiene un ancho total de 9 m; la calzada central tiene 6 m de anchura y está limitada por dos aceras de 1,5 m de ancho dispuestas en voladizo.

La presa está proyectada para su ejecución en bloques. En total se disponen 13 bloques con separación entre juntas de 12 metros, a excepción del bloque central de 15 m de anchura y los extremos. La impermeabilización de las juntas de contracción se realiza mediante dos bandas de PVC de 0,50 m de anchura, cercanas al paramento frontal. Una de estas bandas se prolongará por el paramento de aguas abajo en todas las juntas que quedan sobre la zona del aliviadero. El hormigón de la presa tendrá una resistencia característica a los 90 días de 175 Kg/cm^2.

El aliviadero se encuentra centrado en el cuerpo de presa (en los bloques 0, 1 y 2) y pertenece a la tipología de labio fijo, dimensionado para

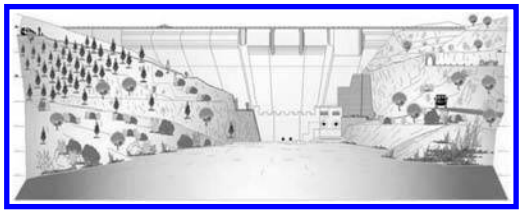




la avenida de proyecto (T = 1.000 años) de 625,9 m^3/s. El aliviadero se encuentra dividido en tres vanos de 10,00 m cada uno. Para ello se han dispuesto dos pilas intermedias de 1,50 m de ancho; los estribos están rematados con otras dos pilas de 3,00 m de ancho, que en su prolongación a lo largo del canal de descarga constituyen los cajeros del aliviadero, los cuales tienen 3,00 m de altura.

Las aguas vertidas por el aliviadero se restituyen al cauce mediante un trampolín que tiene dispuestos dientes deflectores en el extremo del mismo. El cuenco disipador se forma mediante un azud situado unos 70 m aguas abajo de la presa. El azud, construido con el mismo hormigón que la presa, tiene 3 vanos de 10 m de largo constituidos por muros de 3,50 m de ancho con vertedero de labio fijo a la cota 596,00, salvo el central que está 0,20 m más bajo.

La presa existente no dispone de galerías de inspección y drenaje. En las obras objeto del presente Proyecto se han previsto, además de la perforación de una galería sensiblemente horizontal



en la presa antigua, otros dos niveles de galerías en el contacto del paramento antiguo de aguas abajo y el nuevo hormigón de recrecimiento, la superior a cota de demolición de la coronación existente y la inferior a cota de la cámara de válvulas. La sección de todas las galerías es de tipo "baúl" de 2,50 × 1,50 m.

El acceso a las mismas, comunicadas entre sí, se efectúa por sendas galerías transversales, que parten del paramento de aguas abajo de la presa, una de ellas prolongación de la galería de desvío existente. También se puede acceder a las mismas a través de la galería superior con salida a un vial existente en la margen izquierda.

Se ha previsto, para conseguir un monolitismo estructural, la limpieza del paramento actual con chorro de arena, saneo y cepillado mecánico de zonas deterioradas y su drenaje.

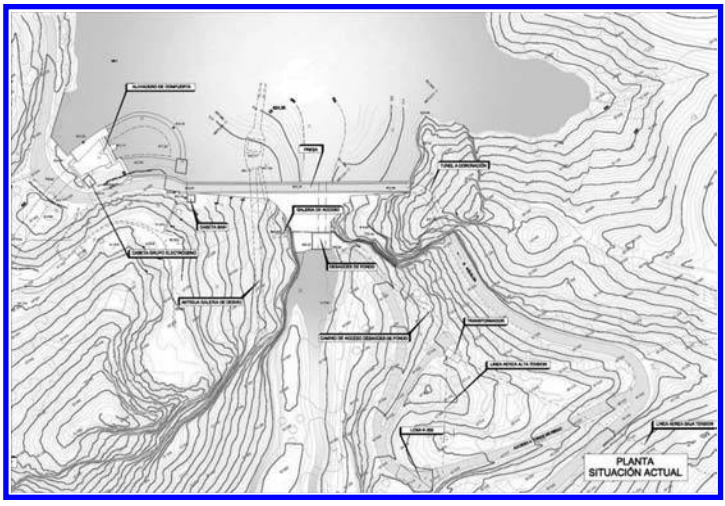
La cámara de los desagües de fondo se encuentra ubicada en el interior del cuerpo de presa y está constituida como un ensanchamiento de la galería longitudinal. La cámara de los desagües de medio fondo se integra con el paramento de aguas abajo, sobresaliendo del mismo con obra de hormigón para mejor integración con la presa.

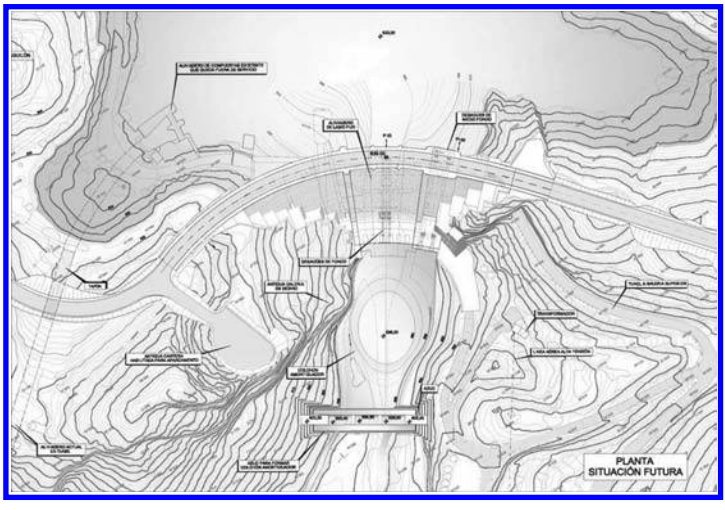
5.2 *Desagües y tomas*

Se prolongan las dos conducciones circulares de 1.000 mm de diámetro de la presa existente, que constituirán el desagüe de fondo, mediante dos conductos rectangulares de 1.000 × 800 mm de acero inoxidable.

Estos conductos desaguan a través del paramento vertical al pie del aliviadero, en el cuenco amortiguador. La cota del eje de los desagües es la 595,00 m con inclinación de 20° a la salida. El sistema de cierre de los mismos está constituido por dos válvulas tipo Bureau de 1,00 × 0,80 m, por conducto. Una hará la función de cierre de seguridad (la de aguas arriba) y normalmente estará abierta, mientras que la otra será la que se maniobrará en las labores de regulación.

Los dos desagües de medio fondo están constituidos por una embocadura con rejilla a la cota 626,50, una compuerta mural de 2,20 × 2,00 m con cámara de accionamiento bajo coronación, una conducción circular paralela al paramento de aguas abajo de la presa de 1.300 mm de diámetro





en acero inoxidable, una válvula Bureau de 1.300 × 1.100 mm y salida mediante una válvula Howell-Bunger de 1.300 mm de diámetro al cuenco amortiguador.

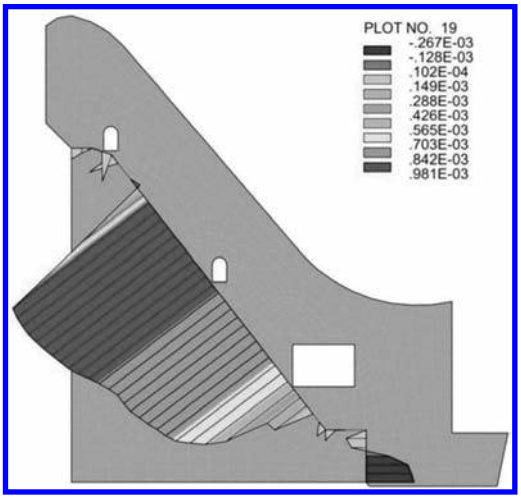
5.3 *Auscultación*

Se ha previsto el control de las siguientes variables, con los equipos indicados:

- Temperaturas (tanto interiores como ambientales): termómetros.
- Subpresiones y filtraciones: piezómetros hidráulicos y de cuerda vibrante, y aforadores de filtraciones con lectura automatizada en cada salida de drenaje.
- Movimientos: medidores de juntas, bases de nivelación, péndulos directos y péndulos invertidos, y extensómetros de varilla.
- Aportaciones y de caudales desalojados: un limnímetro para registro de los niveles de embalse, y una escala adosada a las pilas del vertedero para estimar los caudales desaguados.

5.4 *Medidas correctoras y compensatorias de carácter medioambiental*

En el Estudio de Impacto Ambiental realizado se recogen las limitaciones que el entorno impone a la construcción de las obras y las medidas correctoras y paliativas propuestas. En particular, en el estribo de la margen izquierda se ha detectado la existencia de avifauna protegida, concretamente de águila perdicera, que impedirá la realización de actividades que pudieran derivar en molestias a las mismas durante su periodo de reproducción, comprendiendo esta parada biológica los meses



Análisis del contacto entre hormigones

primaverales. En relación con las medidas compensatorias se ha propuesto, entre otras: estudios de evolución de la población de águila perdicera dentro de la ZEPA de la zona de estudio (parámetros reproductivos, dispersión e instalación de nuevas parejas fuera de territorios conocidos), colaboración con las autoridades ambientales autonómicas para la elaboración del Plan de Recuperación del águila perdicera en Aragón y la realización de un inventario, seguimiento y localización de parejas reproductoras de otras rapaces protegidas en las inmediaciones del embalse de las Torcas.

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Impermeability of the compensation reservoir of HPP "Perućica"

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ABSTRACT: This paper presents the process of embedding polyethylene liners during the reconstruction and rehabilitation of the compensation reservoir of HPP "Perućica". Impermeability of the reservoir aims to achieve waterproofing of reservoir and reduction of water losses to a minimum allowing flexible working of HPP "Perućica" by mitigation of negative consequences of non-stationary phenomena that accompany transitional regimes of the plant. The plan of modernization of HPP "Perućica" defines the installed flow of 81,18 m^3/s which will be achieved by height extension of compensation reservoir and by protection of the inner slope and bottom of the reservoir with waterproof polyethylene foil.

1 INTRODUCTION

Installed runoff amounting to 81,18 m^3/s, (Energoprojekt, 1991, Energoprojekt, 1998) has been defined through the process of modernization of HPP "Perućica". This has caused increase of the volume of the compensation reservoir by achieving final bottom level of 601,80 mnm, and topping the embankment around reservoir for 0,70 m by reinforced-concrete wall, up to the elevation crown of 604,55 mnm. The compensation reservoir is located at the downstream end of Zeta I channel, directly next to the settling tank, before the entrance to the derivative tunnel. Its function is to enable flexible power plant operation in case of abrupt load changes by mitigating negative consequences of unsteady phenomena typical for transient operation modes of the plant. The adjustment of the reservoir to the increased installed runoff of HPP "Perućica" causes the increase of its volume and raising the mediation level from 603,30 mnm to 604,00 mnm. During the exploitation period, i.e. operating mode of the power plant, the phenomenon of karst abysses was registered in the compensation reservoirs, which were regularly rehabilitated, i.e. sealed during overhaul of the power plant. In order to resolve the problem of registered karst abysses in the compensation reservoir and to achieve its water-impermeability, i.e. elimination of water losses, the solution of HDPE foil coating of the reservoir has been adopted.

Design solution involves topping of the peripheral reservoir embankment by reinforced-concrete wall with polyethylene liner protection of the internal slope and bottom of the reservoir.

Bottom of the reservoir (level 601,80 mnm), previously cleaned of mud and silt, in which the rehabilitation of abysses and installation of deaeration pipes were performed, is being coated with foil, as well as embankment slopes up to the vertical part of the wall. Concrete layer of the thickness of 7 cm in rings of 2,5 m reinforced by micro fibers has been put over foil at the slopes of the reservoir. Downstream slope of the embankment has been coated with stone. Water impermeability of the reservoir is achieved in such a way, i.e. elimination of water losses. Deaeration of abysses is achieved through filling the holes with coarse gravel, and up to the depth of 1 m the vertical perforated PVC pipe Ø250 mm is put. This pipe is put in horizontal and vertical by means of elbow insert to the closest main aeration pipe which is also perforated. Main aeration pipes are mutually parallel, 0,40 m embedded into the gravel, placed vertically on the longer part of the reservoir, in total 8 lines at the distance of 60 m. The pipes are directed in parallel to the upstream slope with output through the embankment on the downstream side. The output end of the pipe is protected by steel grating with a lid, while the pipe itself is laid through concrete block of 0,55 m × 0,55 m. After preparation of the bottom foundation and slope of the reservoir, the installation of HDPE polyethylene foil began in order to achieve water impermeability of the bottom of reservoir.

Compensation reservoir with surge tank in Slivlje is given in Figure 1. Polyethylene foils of thickness of 2,5 mm which is water-impermeable and which ensures optimum conditions for installation and minimum field mounting (welding of couplings) is necessary for sealing of the bottom of reservoir.

Foil of thickness of 2,5 mm should be strong enough to ensure resistance to physical damages during transportation to the site and during regular repair of the reservoir bottom (mud cleaning and

Figure 1. Compensation reservoir with surge tank.

Table 1. Physical features of polyethylene foil.

Property		Value	Unit	Norm test
Density		0,94	g/cm^3	ISO-R 1183
Tensile stresses	At classification	18	N/mm^2	ISO-DIS 527
	At tear	24	N/mm^2	ISO-DIS 527
	At 5% elongation	1600/2,5 mm	N/50 mm	ISO-DIS 527
Elongation percentage	At plastification	13	%	ISO-DIS 527
	At tear	800	%	ISO-DIS 527
Elasticity modulus		800	N/mm^2	ISO-DIS 527
Factor of thermal expansion		$1{,}2 \times 10^{-4}$	oC^{-1}	VDE 0304
Water absorption		0.085	%/4 days	ISO-R 62

possible bottom rehabilitation at the point of open abyss).

Also, foil should be resistant to high temperatures, ultra violet rays and aging. Physical features of the foil requested by project documents, i.e. of the installed foil, are given in Table 1 and refer to foil of thickness of 2,5 mm.

Foil manufactured by AGRU—Austria (AGRU, 2009) has been installed at the compensation reservoir and it is of physical features requested by project documents. Foil has been supplied in rolls of dimensions of 7 m × 100 m and thickness of 2,5 mm (77/PE) and each roll has its card containing features of the material, identification of the product, number of roll, length, width, name and address of AGRU, in accordance with technical specifications. Foil whose surface was rough at both sides, has been installed in the embankment slopes of the reservoir, while in the bottom the foil with smooth sides has been installed. Foil has been tested during manufacturing at the factory and supplied with satisfactory criteria in the necessary amount for the area of about 130.000,00 m^2.

2 INSTALLATION AND WELDING OF FOIL

The installation of HDPE (High Density Polyethylene) foil on the prepared foundation has began, roll by roll, enabling overlap necessary for welding of the foil, Figure 2. Installation and welding of the

Figure 2. Installation of foil in the reservoir.

Figure 3. Extruded welding of foil.

foil was performed by licensed welders, members of the foil manufacturer in accordance with DVS 2219 and DVS 2212. Foil is set in the appropriate position by stretching the foil over slopes and bottom of the reservoir, so that overlap from 10 cm to 15 cm necessary for welding is ensured. Welding of the foil is performed extremely carefully and precisely because both sealing of welding seams and functioning of the whole system depend on that. Coupling of the foil is achieved by welding by means of heated wedge or hot air (ZIGMA, 2009), by which case double welding seam with control channel for checking the seam is made. This welding system represents the most secure and the most economical manner for coupling the pieces, since sealing of welding seams may be tested by compressed air through control channel.

This welding technique has the following features:

- Welding is performed without using the filler metal.
- Joint couplings are plasticized by means of heated wedge.
- Coupling force is brought immediately after melting of the joint couplings.
- Welding machine is equipped with compressed system for initiation of coupling forces with double effect.

At some points where due to reservoir geometry welding with heated wedge could not be performed, the extruded welding applied (Figure 3). Parameters of heated wedge welding are the following:

- Wedge temperature: average value of 385°C
- Welding velocity: range from 1,4 to 2,2 m/min, and average value of 2 m/min
- Welding force: average value of 900 N.

Prior to creation of overlap, special care is taken about cleaning of joint couplings before welding in order to ensure clean surface free from dust and all possible dirt. Prior to initiation of welding, the welder performed welding tests under the existing weather conditions. Foil temperature, humidity, wind, etc. represent factors of impact on welding and they are of importance for the selection of appropriate time for welding.

This welding technique has the following features:

- Welding is achieved by usage of the filler metal, which is pressed out of the plastification unit as fibers.
- Welding filler material is homogeneous and completely plasticized (welding rod).
- Joint couplings are being heated to the welding temperature.
- Coupling performs under pressure.

Before the welding process start, edges of a piece or overlap are slanted. Overlapping is at least 10 cm. Heated air which emerges out of the nozzle of welding device heats joint couplings of the parts which are being welded under welding temperature. Welding filler material which constantly emerges out of the manually operated device, embedded into the welding joint. Output mass current moves the device forward and determines the welding velocity, and heating of joint couplings harmonizes with that velocity. No subsequent tempering is necessary for welded couplings. After finalized coupling, visual check performs.

During welding, protocols on welding were taken containing the following data: number of the seam, name of the welder, identification number of the roll foil, type of machine, temperature of air and weather conditions, welding parameters.

3 TESTING THE WELDING SEAMS

Checks of the seams have been performed (ZIGMA, 2009). Each seam has its identification

number. Information about coupling contains data about number of the seam, identification number of the welder, number of the machine, temperature of air and weather conditions, welding parameters, number of roll foil, and date of checking. All stated information is contained in the protocol for the seam which is being checked. Testing of welding seams was achieved with pressurized air. All welding couplings were tested on impermeability. Each welded coupling was first visually checked, taking consideration about the following criteria:

- Faultless state of the control channel,
- Overlap along whole welded length has to be even.

Each welded coupling with control channel is being tested by pressurized air. Control pressure depends on surface temperature of the foil and it amounts to 2,5–3,5 bars, and the average value amounts to 3,00 bars. The average pressure drop during testing is 0,1bar. Both ends of the welded foil are being closed with hot air device. Control needle with control reinforcement is being carefully installed into the air channel, necessary pressure reaches and it is being observed for 10 minutes to 15 minutes, Figure 4.

If there is no pressure drop then welding is water-impermeable. All control apertures at the end of welded coupling are being closed by means of extruded welding. If testing with air indicates negative result, drop of the pressure shall begin and the following measures are to be taken: control of the coupling ends and main control of welded coupling, surrounding zones are being isolated, duration of the testing of welding reduces and the fact of ill-welded part is established.

This is the point of time when extruded welding of the part begins and afterwards the repeated check of that part, as well.

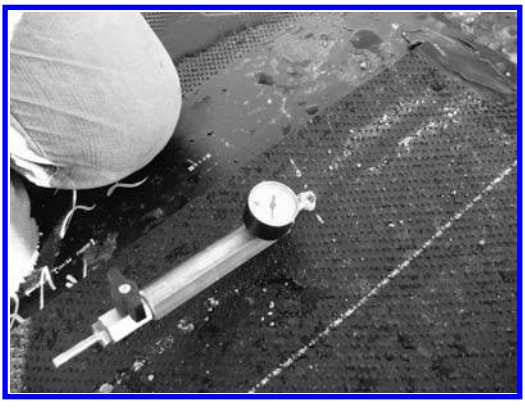

Figure 4. Testing the welding with pressurized air.

4 CONCLUSION

Adjustment of the reservoir towards increased installed flow of HPP "Perućica" caused increase of its volume and raise of its mediation level from 603,30 mnm to 604,00 mnm. Design solution—topping of the peripheral reservoir embankment by reinforced—concrete wall with protection of reservoir internal slope and bottom achieved by polyethylene water-impermeable foil, ensures minimum water losses in the reservoir, as well as flexible operation of the plant in case of abrupt load changes, by mitigating negative consequences of unsteady phenomena typical for transient operating modes of the plant.

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Other related topics

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DamER: Integrated emergency management system in dams

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ABSTRACT: On the basis of experience acquired over years in the implementation of dam emergency plans, a clear need has become apparent for a tool to assist operators in the steps to be taken to ensure an optimal resolution of any incident.

This paper presents an integrated dam emergency management system already implemented at various dams in Spain that includes all the documentation associated with the Plan and the dam, assists decision-making, activates the various scenarios, executes the procedures provided for in the plan, including the activation and deactivation of public warning systems, coordinates all the necessary resources for appropriate action, and provides hydrometeorological forecasting and alert services.

It allows all the actions taken, practice drills and dam technician training sessions to be fully traceable. Operating from the crisis room itself and from hierarchically superior control centres or the facilities of other bodies, this system allows constant maintenance and continuous updating of any modifications that the plan may undergo.

1 BACKGROUND

1.1 *Legislative framework*

Currently there is no single legal text covering all the necessary technical aspects and requirements in this type of hydraulic infrastructure. The legal framework for dam safety is basically set out in the Instruction for the design, construction and operation of large dams (1967), the Basic Directive on Civil Protection Planning for Flood Risks (1994), the Technical Dam and Reservoir Safety Regulation (1996), Law 11/2005 revising various articles linked to dam and reservoir safety in the Water Law 10/2001 on the National Hydrological Plan, and Royal Decree 9/2008 amending the regulations on public water resources adopted by Royal Decree 849/1986. This adds a new title dealing with the safety of dams, reservoirs and basins, with the main aim of unifying safety criteria in a single text, regardless of where hydraulic structures are located or who they are owned by, and defining the responsibilities of the competent authorities with regard to safety.

Moreover the Directorate General for Hydraulic Structures at the Ministry of the Environment and Rural and Marine Affairs drew up a series of technical guidelines supplementary to the previous regulation so as to facilitate the work of dam and reservoir project designers, construction managers and operators, providing information, recommendations and examples to follow.

1.2 *Justification of an integrated Dam Emergency Plan management system*

The purpose of a Dam Emergency Plan (DEP) is to specify the actions to be taken and the organization of human and material resources required to control risk factors that may affect dam safety, and to facilitate emergency preparedness in the services and resources that will have to intervene to protect the population in the event of a burst, failure or other incident.

Accordingly DEPs will contain a large amount of information. By way of example we may highlight dam safety analysis, spatial zoning, an analysis of phenomena that may be triggered in an emergency situation, guidelines for action, documents of each organization, material resources and human resources. Thus it is vital to have a system integrating all this information and the associated resources and helping the operator to take action.

With more than 20 years' experience as an engineering firm specialized in technological solutions applied to water and the environment, Adasa offers a high degree of specialization in DEPs. Given the evident need, Adasa has developed an integrated dam emergency management system: DamER.

2 DESCRIPTION OF THE DAMER SOLUTION

DamER facilitates the management of dam emergencies, covering all the necessary requirements so as to minimize possible risk situations. Its functional modules are as follows:

- Incident Management (IM)
- Communications Management (COM)
- Public Warning System (PSW)
- Decision Aid System (DAS)

With these four modules it is possible to cover all aspects linked to decision-making and emergency management:

- Helping take decisions on the scenario to be activated
- Providing additional real-time meteorological information, forecasts and observations
- Activating and controlling the various emergency scenarios
- Coordinating the available staff and resources
- Carrying out the procedures specified in the Plan for each scenario: communications to the bodies involved, activation/deactivation of public warning systems, monitoring and inspection actions, etc.
- Continually supervising siren control systems
- Ensuring the traceability of all actions for legal purposes

2.1 *Incident Management*

The Incident Management module provides assistance in the DEP's implementation so as to deal with incidents and coordinate the available resources for resolving them. Its purpose is to activate and control the various emergency scenarios, guiding the Plan Director in declaring that a scenario has been entered, and in the warnings and actions to be undertaken, many of which are processed automatically.

This module's tasks are substantially as follows:

- Viewing the state of current incidents and the emergency history
- Activating an emergency scenario according to the phenomena detected
- Executing the procedure provided for in the Plan for each scenario
- Coordinating the resources available for resolving incidents
- Viewing dam documents
- Implementing traceability in all actions

The application is fully web-based, accessible locally or remotely from an area centre or from the facilities of the bodies involved in the emergency. It also allows the module to be run in normal, drill or training mode, enabling operators to be trained and facilitating scheduled drills with Civil Protection bodies.

DamER's Incident Management module is structured in the following menus: inbox with summary of historical events, scenario declaration according to the phenomenon detected, tray of tasks to be performed according to the phenomenon detected, setup menu, documents associated with the dam and DEP.

The system has an internal database allowing all aspects of the DEP to be modelled and related, once transposed from the Plan document:

- Phenomena and their indicators and thresholds
- 5 possible states: Normal, Scenarios 0, 1, 2, 3 and end of emergency
- Actions by scenario (communication actions, warnings and alarms or specific actions associated with a phenomenon)
- Forms predefined in the DEP
- Own and external material resources
- Contacts, external human resources assigned to the Plan (directories of bodies linked to the DEP, material resource managers, etc.)
- Users and profiles: Plan Director, area managers, operators, etc.

Figure 1. DamER modules.

Figure 2. Incident management module.

2.2 *Communications Management*

The Communications Management module is integrated in the same web application. It links warning tasks corresponding to the current scenario with contacts in the DamER address book and communications devices.

In fax communications, DamER automatically fills out the form, also allowing comments to be entered, sends the fax to eh corresponding contact and reports and the transmission status.

In oral warnings, DamER shows the operator the script of the message to be sent according to the current scenario. It makes the phone call to the contact and displays the call status. Thus DamER automates fax-sending and call setup tasks, whereas the final confirmation of satisfactory communication with the body receiving the alert is given manually by the operator.

Figure 3. Integration of communications equipment.

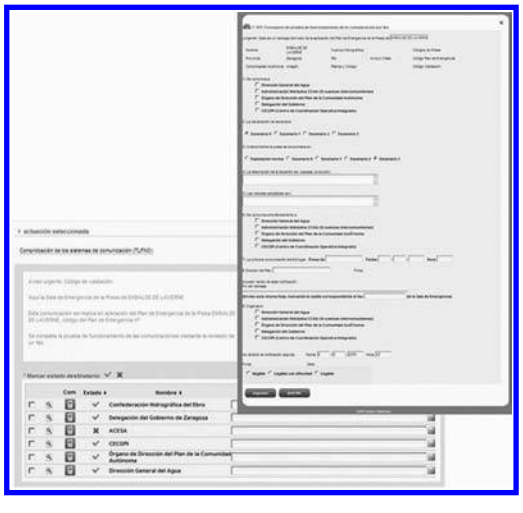

Figure 4. Fax and oral communications forms.

2.3 *Public Warning System*

In emergency scenario 4, the Plan Director must decide whether to activate/deactivate the warning sirens. For this purpose he/she uses the graphic interface in the Public Warning System (PWS):

- Geographic representation of siren location
- Schematic representation of each siren
  - Control of siren activation/deactivation
    - Selection of sirens to be activated/deactivated
    - Siren test in silent mode
    - Activation/deactivation command

The DamER solution integrates both electronic and pneumatic sirens. It continually supervises the state of siren stations, facilitating maintenance tasks and ensuring that everything is operational at the moment of activation:

- Facility access control
- Supervision of the pneumatic circuit or electronic siren driver
- Outages in electricity supply or communications
- Alarm logging and acknowledgment

Finally the DamER PWS module was designed taking account of the mission-critical service criteria typical of an emergency management system: secure power supply, redundancy in communication channels (TETRA/GPRS, UHF/GPRS, etc.), security mechanisms to prevent inadvertent activation of sirens, protected local security button, etc.

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2.4 *Decision Aid System*

DamER provides web-based weather forecasting and observation services in the whole country for the purpose of facilitating decision-making, both in adverse situations and in normal operating tasks:

- 6-day weather forecasts
- Real-time weather radar and lightening strike display

Figure 5. Orellana dam draining (Guadiana River Basin Authority).

Figure 6. Decision aid system: real-time.

Figure 7. Decision aid system: 6-day forecast on mobile device.

- Short-term rainfall forecast: 2h nowcasting
- Other daily rainfall accumulation products
- Configurable text message alert system

2.5 *Notable advantages*

DamER offers a series of functional and operational benefits, notably including the following:

- Providing flexibility and a progressive scale in the loading of Plan elements
- Including operating modules allowing the implementation of planned practice drills and operator training
- Allowing for the entry of all kinds of phenomena indicators (quantitative and qualitative)
- Establishing that the declaration of a scenario is the responsibility of the Plan Director user, but allowing other warning or inspection actions to be delegated to other operator-type users
- Automating, as much as possible, actions for preparing and sending official warning forms, always with final confirmation from the person responsible
- Ensuring total traceability, permanently logging the various actions taken and allowing user comments to be entered
- Facilitating viewing of Plan and dam documents: drawings, detailed procedures, etc.

But above all it is worth noting that DamER allows DEP information to be kept up to date, suited to the reality and to the experience progressively acquired in emergency management. These goals could hardly be achieved using DEP procedures printed on paper.

From the technical viewpoint, the advantages of DamER are:

- It has a fully intranet-compatible design:
  - Allowing remote management of the Plan, from regional or area centres, providing full functionality as long as they have an intranet and the necessary user privileges
  - Allowing remote consultation from the facilities of the bodies involved in the emergency, facilitating crisis management visibility
- Designed as a mission-critical system, supporting high-availability configurations
- Based on Open Source and multiplatform technologies, with possible versions with Linux or other operating systems
- Multilingual support, with an interface that can be adapted to any language

3 PRACTICAL EXPERIENCE AND OPEN QUESTIONS

Some real experiences and best practices are compiled in this chapter. They came basically from the implementation of 18 DEPs in the Ebro, North, Guadalquivir, Guadiana and Tajo River Basins, performed by Adasa Sistemas, and some recent general flood episodes successfully managed by Guadiana River Basin.

This accumulated practical experience allows us to see that there are still many issues to be resolved in order to find a compromise solution between regulatory, technical and organizational aspects. Different projects have highlighted various issues and open questions which show that there is still some way to go if we are to be able to put technological solutions at the service of organizational structures and to adapt to the reality of operation and crisis situation management.

Over the projects conducted, some questions have arisen: Can the dam's sirens be activated from a distance? What happens if there are simultaneous incidents at several dams? Or can incidents be managed remotely? The answer is: it is technically possible, but it seems operationally unviable. DEPs specify countless local inspection tasks involving

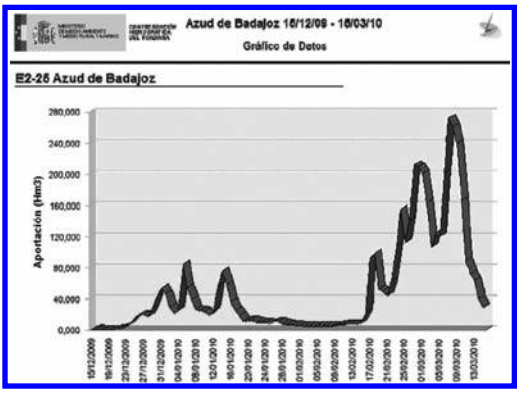

Figure 8. Guadiana study case.

the constant presence of staff at the dam, as from Scenario 0.

So it seems reasonable for solutions to evolve towards a distributed management response to an emergency affecting a group of dams, with the Plan Director working from a regional centre, but with staff posted at each dam.

One successful case may be described as example to reinforce this distributed management strategy. From 15th of December up to 15th of March, Guadiana River Basin faced a persisting ordinary flood event, with 3 important flood peaks. The storage water grows up by twice in 2 months, from the initial 2935 Hm^3 up to the final 6500 Hm^3. The episode did not reach the Scenario 0 of emergency, but the organizational aspects were fully similar.

During the main flood peaks, operational staff was on 24 h duty on damps like Cijara, Garcia Sola, Orellana, La Serena, and so on, while the manager director was coordinating from a regional control centre. Also the maintenance team of the automatic hydrological information system was on 24 h duty in order to guarantee that the communication and information systems were fully operative. Since the episode was persistent, from some periods the manager director kept the situation monitored from home, using automatic hydrological information system's extranet facilities.

Other questions arise concerning the possibility of automatic scenario declaration. But a glance at the specified phenomena shows that the indicators are for the most part qualitative, not measurable. Accordingly the few quantitative thresholds defined in DEPs do not correspond to a simple algebraic formula, liable to be automated. Moreover, some not desirable 'false positives' alarms may came from sensoring devices. Finally, common sense tells us that the system is able at most to suggest which scenario to activate, but the final decision is human, under the Plan Director's responsibility.

4 OPORTUNITIES FOR ORDINARY DAM OPERATION

Finally, on the basis of actual experience in emergency drills it is clearly very hard and indeed virtually impracticable to manage an actual emergency situation with the sole aid of DEP procedures printed in a manual. The large amount of information associated with DEPs, the need for an open system allowing their content to be constantly updated, the short response time involved in emergency situations and the requirement of achieving full traceability all lead to a demand for the application of new technologies in emergency management, though also in normal operation. There is a widely held view that a management tool such as DamER is truly indispensable.

Some potential problems may arise, mainly because emergency communications infrastructures and support systems will be only used in rare episodic situations elapsed in time. And in these episodic situations, the emergency systems should be fully operatives. In order to achieve this objective, a system monitoring, maintenance and operators training strategy must be implemented. That means adding a maintenance cost to an already costly emergency infrastructure.

But, there are alternative ways to face this maintenance problem, and try to convert it into dam management opportunities. The main idea is to keep the emergency systems in a constant 'warm up' mode by assimilating the normal exploitation tasks over the emergency infrastructures. Such continuous use will keep 'fit' the emergency infrastructure.

Following are some best practices examples.

Dam emergency infrastructures include dual communications channels with enough wideband for voice use. These channels may be proprietary infrastructure or hired communications services. In any case, those channels may be used for normal dam operations. As example, Guadiana River Basin is deploying wideband communications services for main reservoirs and expanding it trough local dam wireless networks. The aim is to fulfil the DEPs communications requirements, while supplying added value services for the normal dam exploitation tasks. This integrated approach includes:

- Expanded voice and data Wimax services in dams, like Zújar, with a poor GSM coverage, even expansion to internal tunnels
- Video services for secure operations and vandalism prevention

- Integration of sensoring and telemetry systems: automatic hydrological information system, dam structure monitoring (movement, deformation and stress sensors)
- Wireless facilities for mobile devices (handhelds, laptops, etc) for maintenance operations
- Remote Intranet access to corporative information and tools

As other potential reusable solution is the DamER platform and applications infrastructure. DamER is designed to guide the incidents management during the dam emergency situations. But it can be easily adapted in order to guide the operational procedures during normal dam exploitation and ordinary flood management situations.

5 SUMMARY AND CONCLUSIONS

The large amount of information associated with DEPs, the need for an open system allowing their content to be constantly updated, the short response time involved in emergency situations and the requirement of achieving full traceability all lead to a demand for the application of new technologies in emergency management, though also in normal operation.

Starting a new generation of information systems, DamER is offered to the market as a technological innovation to facilitate optimum action in an emergency. Parametrizable according to the needs of each dam, it supports interoperability with other manufacturers and integration in larger-scale systems. It may operate from a dam control centre, in a group of dams or from the facilities of other bodies involved in resolving the situation.

ACKNOWLEDGEMENTS

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Hacia una gestion integral de la seguridad de las presas de la confederación hidrográfica del duero

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RESUMEN: El entendimiento y el reconocimiento de los distintos factores de riesgo presentes en las actividades relacionadas con la seguridad de presas y embalses constituye la base conceptual para la confección de sistemas lógicos o modelos orientados a informar la toma de decisiones en seguridad de presas. El propio Reglamento del Dominio Público Hidráulico (RD 16 de Enero de 2008) recoge la necesidad de contemplar la gestión del riesgo como un elemento fundamental de la Seguridad de las Presas, siguiendo el ejemplo de algunos de los países más desarrollados del mundo. En esta línea de trabajo, la Confederación Hidrográfica del Duero está llevando a cabo una serie de actividades orientadas a crear un sistema de gestión integral de la seguridad de sus presas con el objetivo de que dicha gestión sea más eficiente, moderna, transparente y socialmente válida para la toma de decisiones. El contenido del artículo parte del contexto legislativo y la evolución conceptual en el entendimiento del riesgo de inundación, para después desgranar las labores de gestión de riesgo vinculadas a la seguridad de presas y embalses que están siendo desarrolladas por la Confederación Hidrográfica del Duero, con especial hincapié en el cambio de rol asumido por este organismo y la necesidad de integración de la comunicación como un pilar de la gestión del riesgo

ABSTRACT: Understanding and recognizing all different risk components that are inherent in dam and reservoir safety management constitutes the conceptual basis to implement logic systems or models aimed to inform decision making. The most recent Spanish legislation on dam safety (Reglamento del Dominio Publico Hidraulico; Real Decreto, January 16th, 2008) incorporates the need of risk management as a fundamental component of dam safety issues in a modern country. In this context, the Duero River Authority (Confederacion Hidrografica del Duero) is undertaking and ambitious program in order to implement a more efficient, modern, transparent and socially accepted managing system. The article starts by reviewing the legal context and the evolution in flooding risk understanding and continues by describing all the activities that are currently under development at the basin, with special focus on the new role that Duero River Authority is assuming and the need of integrating communication activities as risk management pillar.

1 INTRODUCCION: GESTION DE RIESGO DE INUNDACIÓN Y COMUNICACIÓN

Los conceptos de gestión de riesgo de inundación y comunicación son indisociables el uno del otro, debido al propio concepto de riesgo y a su carácter intrínsecamente dinámico.

Comenzando por el "concepto" de riesgo de inundación, se entiende como tal la combinación de la probabilidad de ocurrencia de inundaciones con las consecuencias asociadas a dicha situación. En segundo lugar, el entendimiento de ambos factores (probabilidad de ocurrencia y consecuencias), explica de por sí cómo el riesgo es un concepto dinámico, dado que los factores que lo integran varían inexorablemente con el tiempo.

El reconocimiento efectivo de la doble componente del riesgo de inundación a nivel legislativo

puede afirmarse que se produce en España en el año 1995, con la publicación de la "Directriz Básica de Planificación de Protección Civil ante el Riesgo de Inundaciones" (BOE de 14 de Febrero de 1995).

Más allá de esa importancia concedida a las consecuencias de la inundación, la Directriz de 1995 impone a los propietarios de presas nuevas obligaciones derivadas del entendimiento completo del riesgo. Si hasta la década de los noventa los propietarios de presas se ocupaban fundamentalmente de gestionar la probabilidad de fallo de sus estructuras (mediante el cumplimiento de códigos como la Instrucción para el Proyecto Construcción y Explotación de Grandes Presas de 1967), a partir de 1995, con la obligación de redactar e implantar Planes de Emergencia, adquieren responsabilidades en la gestión de las consecuencias.

Esta nueva realidad o "cambio de paradigma" requiere necesariamente de acciones de comunicación, que en un primer nivel se engloban dentro de la gestión de las emergencias, a través de los mencionados Planes de Emergencia de Presas.

Este cambio de paradigma constituye un fenómeno de carácter internacional. Son numerosos los países que durante los años noventa incorporan legislación general de protección frente a inundaciones y específica de seguridad de presas. En ella se define la gestión de emergencias y se da un peso específico importante y explícito a las tareas de comunicación.

En España, el Reglamento Técnico de Seguridad de Presas y Embalses de 1996, haciéndose eco de esta realidad, recoge todas las prescripciones que en materia de Planes de Emergencia de Presas se habían promulgado un año antes en la Directriz Básica de Planificación de Protección Civil ante el Riesgo de Inundaciones. Además, da un paso más allá, estableciendo que el criterio básico que debe guiar las actuaciones en seguridad de presas es el "riesgo potencial asumible".

Si bien cabe discutir algunos aspectos de dicha definición, como lo redundante del carácter potencial del riesgo, o el término "asumible" (la comunidad internacional entiende hoy que el riesgo sólo puede ser "tolerable"), constituye una profundización en dicho "cambio de paradigma" y amplía, si cabe, el espectro del nuevo papel de la comunicación en dicho contexto.

De hecho, los propios términos de "riesgo", su "tolerabilidad", la implicación de cómo se comunican dichos conceptos a la población e incluso el rol necesariamente distinto que adquieren los propietarios de presas respecto del resto de la sociedad por su nueva necesidad de comunicar y comunicar bien para llevar a cabo una gestión efectiva y eficiente del riesgo, constituyen un enorme reto.

2 ANALISIS DE RIESGO Y SEGURIDAD DE PRESAS

Hasta la fecha, por razones históricas y técnicas, la seguridad de presas y embalses había estado encomendada al enfoque ingenieril tradicional, consistente en controlar los riesgos mediante el seguimiento de reglas y procedimientos sancionados por la práctica así como verificar coeficientes de seguridad, entendidos como una medida conservadora y prudente.

En particular, la necesidad de desarrollar una técnica de análisis de riesgos a partir de la cual llevar a cabo una gestión moderna, eficiente y transparente surge a partir de una serie de condicionantes como los que se describen a continuación:

a. El envejecimiento del parque de presas (la mayoría de las estructuras supera los treinta años de antigüedad y, un porcentaje muy amplio los cincuenta años en explotación), así como la diferencia entre el conocimiento ingenieril actual y el existente cuando éstas fueron diseñadas y construidas.
b. La demanda de mayores niveles de seguridad para la población y los bienes ubicados aguas abajo de las presas.
c. La creciente demanda respecto de una mejor justificación del uso de los fondos públicos, incluyendo los programas de seguridad de presas.
d. La necesidad de priorizar acciones correctoras para conseguir la mayor y más rápida reducción de riesgos posible.
e. La práctica imposibilidad de construcción de nuevas estructuras por aspectos fundamentalmente sociales y medioambientales.
f. La necesidad de optimizar la gestión de sistemas de recursos hídricos así como de aumentar la capacidad de regulación de los mismos para dar respuesta a una demanda de abastecimiento creciente y en una situación de aparente incremento de eventos climatológicos extremos (avenidas y sequías).

En este contexto de necesidad de mantenimiento de las infraestructuras hidráulicas, mejora de la explotación y aumento de la capacidad de regulación, resulta de especial importancia la estimación de riesgos relativos a la propia seguridad de la estructura así como los referidos a la capacidad de satisfacción de las demandas. Aún más, resulta imprescindible identificar niveles de riesgo tolerables y establecer programas integrales de gestión de

la seguridad, incorporando indicadores de eficiencia en las inversiones.

Todo ello implica, a día de hoy, una continua inversión en investigación y desarrollo (por ejemplo incorporación de factores antrópicos y medioambientales de riesgo, mejoras continuas en las estimaciones y análisis de incertidumbre, desarrollos de software, etc.). Por tanto, puede decirse que el análisis, evaluación y gestión de riesgo requiere también de una componente formativa e investigadora continua.

La Figura 1 trata de representar el papel de los modelos de riesgo como herramientas que permiten integrar toda la información existente proveniente de las prácticas actuales en seguridad de presas para después informar la toma de decisiones. Además, refleja de forma explícita los vínculos con los documentos legales españoles con las necesidades de conocimiento para configurar modelos de riesgo.

Como se observa en la figura anterior, el proceso de la gestión integral es esencialmente dinámico y conlleva una actualización continua de los modelos de riesgo y del Archivo Técnico.

Así, si bien cualquier "estándar" o código ingenieril tiene de por sí una limitación binaria ("cumple" vs "no cumple") y estática (cuando "se cumple" sería el final del proceso), los modelos de riesgo capturan el carácter continuado ("cumplir" sería un punto de partida y a partir de ahí hay que seguir trabajando para controlar el riesgo permanentemente) de las tareas de seguridad de presas.

Figura 1. Vínculos entre los modelos de riesgo, documentos legales, áreas de conocimiento y la gestión integral de seguridad de presas (Serrano et al., 2010).

Y es precisamente ese carácter continuo de gestión de riesgo el que requiere y pone en valor a la buena comunicación como un elemento clave de control de riesgo.

Incidiendo en este aspecto cabe remarcar que, incluso cumpliéndose estrictos estándares de seguridad, el hecho de no comunicar implicaría un aumento del nivel de riesgo, nivel de riesgo que es cuantificable y monitorizable mediante modelos de riesgo apropiados.

3 VISION Y APLICACIÓN DE LA GESTION INTEGRAL DE SEGURIDAD DE PRESAS EN LA CONFEDERACIÓN HIDROGRÁFICA DEL DUERO

La Confederación Hidrográfica del Duero (CHD) está llevando a cabo una serie de tareas complementarias a su programa de seguridad de presas basadas en las técnicas de análisis de riesgos, a imagen de otros organismos internacionales equivalentes, como el Bureau of Reclamation (USBR) o el Cuerpo de Ingenieros de los Estados Unidos (USACE).

La sistemática a través de la cual la CHD está incorporando el análisis y evaluación de riesgos a la gestión integral de la seguridad de todas sus presas y embalses se articula a través de tres actividades básicas:

a. Actividad A: Análisis y Evaluación de Riesgo para la presa y embalse de Castrovido, actualmente en construcción (Sanz et ál., 2010).
b. Actividad B: Identificación de modos de fallo de cada una de las presas como parte de las actividades de los trabajos de mantenimiento y conservación de los sistemas actualmente en explotación (Fernández Baños et ál., 2010).
c. Actividad C: Elaboración de los modelos de riesgo de los sistemas de presas como ayuda para una gestión integral de la seguridad de presas y embalses (Ardiles et al., 2010).

El conjunto de actividades iniciadas en el campo de la gestión integral de la seguridad de presas por la Confederación Hidrográfica del Duero tienen por objeto facilitar y mejorar dicha gestión, de manera que:

a. Las actuaciones que se acometan estén justificadas en términos de eficiencia en la reducción de riesgos.
b. Los procesos de justificación y toma de decisiones sean transparentes, defendibles y socialmente válidos.
c. Se tomen las mejores decisiones posibles, particularmente en aspectos de inversión.

Además, en concordancia con la importancia de los aspectos de comunicación en la gestión de la seguridad de presas y embalses y, con la vocación de integrar todos los aspectos relativos a la comunicación dentro de un sistema de gestión integral de seguridad, la CHD está trabajando en el estudio del impacto de la comunicación como una medida "no estructural" de reducción y gestión de riesgo así como en la propia necesidad de comunicar correctamente con la población para evitar que el nivel de riesgo por falta de implicación o de conciencia aumente con el tiempo.

De hecho, la comunicación eficaz con la población disminuirá el riesgo si, utilizada de manera continuada, en cada una de las etapas de la gestión de riesgo, se genera credibilidad de las autoridades hacia los públicos afectados por una posible inundación.

A su vez, la comunicación generará credibilidad si la información que se aporta es transparente, bien documentada, se explica con naturalidad y empatía y se realiza de manera continuada. La transparencia requiere de un Plan de Comunicación con el objetivo u objetivos bien definidos, los públicos bien identificados, las estrategias adecuadas para cumplir los objetivos, los mensajes claros, sencillos y bien documentados y los lenguajes adecuados a cada público.

Como ejemplo de los conceptos y aspectos en los que se está comenzando a trabajar, la disminución del riesgo de pérdida de vidas humanas en caso de inundación, depende de la correcta aplicación del Plan de Emergencia de la presa por parte de las autoridades y gestores de la misma y del correcto seguimiento de las instrucciones por parte de los públicos afectados.

A su vez, el correcto seguimiento de los públicos afectados depende de un eficaz diseño y aplicación del Plan de Comunicación del Plan de Emergencia de la presa y el cumplimiento de los objetivos del Plan de Comunicación depende de su eficaz aplicación y de la credibilidad de las autoridades que aplican el Plan.

En cualquier caso, la "hoja de ruta" presentada por la Confederación Hidrográfica del Duero trasciende del propio alcance de los modelos de riesgo o de cualquiera de las otras actividades descritas en este artículo, implicando de por sí un nuevo rol de la propia Confederación.

Parte de esta trascendencia está vinculado al proceso en sí, que va más allá de la metodología seguida para confeccionar modelos de riesgo o de los criterios de tolerabilidad de riesgo que se adopten o se desarrollen ex profeso para la cuenca del Duero o, en su caso, para el contexto español.

Como ejemplo de estas implicaciones del propio proceso, la Confederación tiene responsabilidades claras en cuanto a acciones "estructurales", ya sea en términos de ejecutar nuevas estructuras de protección o asegurar la integridad de las existentes. Sin embargo, sobre todo en lo referido a actuaciones no estructurales, como avisos, evacuación o planificación urbanística, las responsabilidades son en general compartidas o exclusivas en algunos casos de otros organismos y autoridades. Por tanto, uno de los retos de llevar a cabo procesos como el que acaba de emprender la CHD es precisamente la generación de consensos y la identificación e implementación, en colaboración con otros organismos involucrados, de aquellas medidas que de forma más eficaz y eficiente sirvan para controlar y mitigar los riesgos en el entorno de nuestros cauces.

La Figura 2 muestra como se estructuran e integran todas las acciones en marcha, puede observarse cómo el "primer ciclo completo" de estimación de riesgo en el sistema estaría concluido a finales de 2011.

Como se desprende del propio esquema, a partir de 2012, todas las actividades de los mantenimientos, las mejoras en predicción conseguidas por el SAIH, la caracterización del comportamiento con la mejora de la auscultación, la forma de explotar las presas, la implementación de los planes de emergencia, las acciones de comunicación o las recomendaciones de los informes de seguridad, podrán ponerse en valor por su contribución a la mitigación del riesgo y, en escenarios de restricción presupuestaria, priorizarse en términos de eficiencia en dicha mitigación. Por último, se prevé extender todas estas actividades, a partir de 2014, al conjunto de las infraestructuras de la Confederación Hidrográfica del Duero.

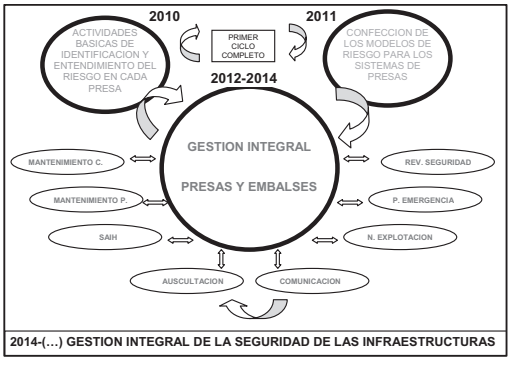

Figura 2. Hacia una gestión integral de la seguridad de las infraestructuras de la Confederación Hidrográfica del Duero.

4 EJEMPLOS NACIONALES E INTERNACIONALES DE APLICACIÓN DEL ANALISIS DE RIESGOS PARA LA TOMA DE DECISIONES

Los aspectos de "gestión de la seguridad", a partir de decisiones informadas sobre "modelos de riesgo" o, de forma más genérica, análisis de riesgo, comienzan a a mediados de la década de los 90. A partir de las publicaciones del Dr. David Bowles (UTAH State University), distintos organismos de seguridad de presas constituyen grupos de trabajo para el desarrollo de metodologías que permitan evaluar riesgos y tomar decisiones de inversión en seguridad de presas basadas en la aceptabilidad o no de los mismos. En particular, los focos de desarrollo inicial se localizan fundamentalmente en Australia (University of New South Wales, ANCOLD, etc.), Canada (BC Hydro, etc.) y Estados Unidos (UTAH State University, Bureau of Reclamation, etc.).

Desde 2005, el Cuerpo de Ingenieros de los Estados Unidos (USACE) y el organismo regulador de la energía hidroléctrica en dicho país (FERC) han puesto en marcha sus propios procesos de gestión de la seguridad de presas sobre la base de información de riesgo, mientras otros países como Francia, en 2008, han promulgado leyes específicas para llevar a cabo llevar a cabo el proceso a escala nacional.

Volviendo al contexto actual español, el hecho de que en los últimos años se hayan concluido la mayor parte de los Planes de Emergencia y de los informes.

De Primera Revisión y Análisis General de la Seguridad y se esté en disposición de planificar y ejecutar las pertinentes inversiones en actuaciones correctoras, constituye un estímulo para poner a disposición de los propietarios de presas nuevas herramientas de ayuda a la toma de decisiones que permitan contrastar la eficiencia de dichas inversiones.

Como ejemplo práctico de aplicación en España, la Confederación Hidrográfica del Duero (CHD) está llevando a cabo una serie de tareas complementarias a su programa de seguridad de presas basadas en las técnicas de análisis de riesgos, a imagen de otros organismos internacionales equivalentes (p.e. US Bureau of Reclamation).

Culminada una primera etapa de cribado o "screening", se desarrolló un análisis completo de los riesgos inherentes a la gestión de seguridad de las presas de los sistemas Carrión y Pisuerga, incluyendo estimaciones cuantitativas de probabilidad de ocurrencia de distintos eventos, la identificación de los potenciales modos de fallo, la probabilidad de fallo asociada a cada uno de los mismos y las consecuencias resultantes.

Los trabajos mencionados han sido recogidos en diversos artículos (Ardiles et ál., 2008) y presentados en distintos foros: Valladolid en 2009, con ocasión de la celebración de la Jornada Técnica de Análisis y evaluación de Riesgos organizada por la CHD, Brasilia en 2009, con ocasión de la 23 Edición del Congreso Internacional de Grandes Presas de ICOLD y más recientemente en Sacramento, en Abril de 2010, donde ha sido seleccionado como uno de los cinco casos de estudio de confección de modelos de riesgo por el Workshop internacional de "Case Histories in Dam Safety Risk Assessment, organizado por la Sociedad Norteamericana de Presas (USSD).

En resumen, puede afirmarse que el alcance y la profundidad conseguida en el conjunto de los trabajos realizados para el Sistema Carrión estaría claramente por encima del promedio del estado del arte actual, pudiendo citarse como un caso de "buena práctica internacional".

Además, la forma de estimar el riesgo global en el sistema Carrión y localizar la porción del mismo que corresponde a cada presa, constituye un caso innovador en este tipo de estudios tal y como pudo constatarse en las discusiones mantenidas en dicho foro (Sacramento, 2010).

La principal innovación consiste en conectar las presas de un mismo sistema, estimar el riesgo global y, para determinar qué riesgo introduce cada presa en el sistema, realizar una segunda estimación considerando dicha presa en particular como "irrompible". Así, la diferencia entre la primera y la segunda de las estimaciones descritas sería el riesgo que cabe achacar a una presa en concreto, entendida la presa como un componente de un sistema más complejo.

De hecho, a partir de los resultados del modelo pudieron establecerse una serie de consideraciones respecto a cuáles de las medidas recomendadas en los informes de seguridad eran más eficientes en cuanto a reducción de riesgo así como respecto de la propia explotación de la presa. En particular:

a. Respecto de las medidas recomendadas en los Informes de Seguridad de las presas de Camporredondo y Compuerto, todas quedaron justificadas en términos de eficiencia, si bien, los resultados permitieron recomendar como priorizarlas: la más eficiente resultó la rehabilitación del sistema de drenaje en ambas presas, seguida de la implantación del Plan de Emergencia y de la rehabilitación de los equipos electro-óleo-hidráulicos.

b. Por otra parte, como sistema, se observó que existe margen de transferencia de riesgo desde Camporredono hacia Compuerto, presa con un riesgo muy inferior a los estándares de tolerabilidad internacionales más exigentes.

Este ejemplo permite visualizar algunos de los principales objetivos que se persiguen mediante la confección de modelos de riesgo para todos los restantes sistemas de explotación en la CHD: disponer de una herramienta de gestión que ayude tomar decisiones de inversión (fundamentalmente programar y priorizar actuaciones) así como permitir un continua monitorización y control del riesgo.

Otros ejemplos de organismos y empresas españolas que están desarrollando proyectos similares desde 2009, éstos en fase de investigación o de aplicación piloto, son Iberdrola y la Agencia Catalana del Agua.

Cabe finalmente reseñar que una de las aportaciones más significativas de la II Semana Internacional sobre la aplicación del análisis de riesgos a la seguridad de presas (Universidad Politécnica de Valencia, 2008), consistió en la divulgación de los resultados del proyecto "A European Methodology for the Security Assessment of Dams" (European Commision. Directorate General Justice, Freedom and Security. JLS/2006/EPCIP/001.), basada en la aplicación de las técnicas de análisis de riesgos a la evaluación de la seguridad de presas frente a acciones de sabotaje, vandalismo y terrorismo. La tendencia mundial en esta materia consiste en incorporar esta rama como una más en la gestión de la seguridad de presas y embalses.

A este respecto, la experiencia del primer y cuarto autor de este artículo como observadores en las inspecciones piloto frente a riesgo antrópico llevadas a cabo por el Department of Homeland Security en el sistema de la cuenca del río Columbia (Washington State, USA), sentó las bases para un ambicioso planteamiento investigador en la materia de manera que pueda integrarse dicho análisis con el resto de actividades actualmente en marcha en la Confederación del Duero.

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Toward comprehensive flood management within dams an experience on Golestan and Boostan dams-Iran

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ABSTRACT: There are many components on flood management, which are classified to structural and nonstructural approaches. The dam is one of the structural components reduce the peak of flood in downstream. During the summers of 2001 to 2005 there occurred some big floods in Golestan and Boostan dams' basin in Iran, Which 2001 flood was historical and biggest one and those were disaster. After that huge flood there had been done three major measures simultaneously. First, rehabilitation of damaged part of Golestan dam, second, revising design flood in Boostan under construction dam and the last measure, to prepare comprehensive flood management plan. The paper presented result of above measures as an experience and is proposed an integrated flood management approach to consider some necessary measures in both dam, down and upstream of dam to dams and people security.

1 INTRODUCTION

There are many components on flood management in river basin, which classified to structural and non structural approaches. The dam is one of structural component which reduce the peak of flood in downstream. The construction of dams usually changes the flow and sediment's regime of river. The capacity of river is changed during the reach to downstream. There are usually managed the river by several sectors and so on, in design of dam must be considered both safety of dam and its downstream as comprehensively. Usually this important subject is not considered by designers. For example in many of dams spillway capacity is more than river capacity in downstream and there is not any conservation and preparedness plan, both structural and non structural especially for city and villages in downstream. Then there is an integrated and comprehensive flood management plan is necessary especially if several dams existed as cascade. Furthermore usually there are existed some problem in dam's operation and exploitation in the time or after flood and there is not a plan on that, and so to prepare rehabilitation plan of dam after flood is necessary. In this paper, an comprehensive flood management plan and some measures on Golestan and Boostan dams after huge floods in Gorgan river basin in Iran is presented.

2 CAS STUDY

The Golestan and Boostan dams are located on Gorgan river basin in Golestan province. This province is located in the northern part of Iran in the south east of Caspian Sea, Figure 1. The Golestan province is one of the flooded area in Iran and has been frequently affected by the disasters of flood and Gorgan river basin especially Golestan dam basin is one of the disaster affected

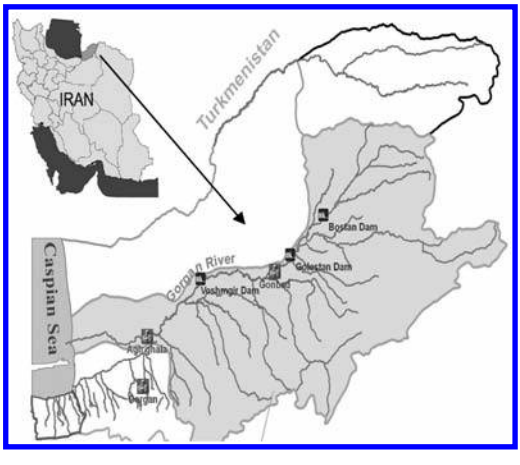

Figure 1. Map of study area.

area in this province. In the last 30 years, a lot of floods have been occurred in Golestan dam basin, which impact of each flood was differ from another floods depend on their intensity. In recent years during 2001 to 2005, there have been occurred four huge floods in Golestan dam basin and was entered a lot of damages follow those. about more than 300 peoples were killed, overall 271000 people were affected, of which 10000 were made homeless, more than 4000 building were heavily damaged and road networks within the province and linking other province were seriously damaged and more than 15000 ha of farmland and 1000 ha of forest and rangeland were destroyed, (UNDP, 2001). Financial losses due to floods 2001, 2002, 2005 were US $77, 36, 69 million respectively. All damages occurred in up stream of dams, Table 1.

1.1 *Features of area*

The study area covers eastern part of Gorgan river basin, Figure 1. The Gorgan river basin is a sub basin of Caspian Sea and flow from east to west finally reached to Caspian Sea. the basin including three parts, based of topographic aspects: 1- alluvial plain with elevation of –16 to 100 m above m.s.l, this region widely utilizing for farming, 2- hilly region with elevation 100 to 400 m which this region utilizing for dry farming and 3- grazing and mountainous region with elevation 400 to 3000 m which covered by forest and grazing. the climate is warm and semi humid and annual precipitation is between 350 to 1000 mm per year in different places and average is 550 mm. Average annual runoff in Gorgan river basin is 880 mcm. There are exist three reservoir dams such as Boostan, Golestan and Voshmgir dams, which are located as cascade up- downstream on main river, two reservoir dam are under construction and three dams are under study. Furthermore there are several small dams, such as rubber dam and other small establishments, too. The area of Gorgan river basin is 11000 sq.km and area of Boostan, Golestan and Voshmgir dam basin are 1500, 5000 and 8000 sq.km, respectively. Average annual runoff in Boostan dam is 45 mcm, Golestan dam 200 mcm and Voshmgir dam 500 mcm and their elevation are 150, 70 and 45 m.s.l., respectively. A unique feature of Golestan dam basin is the Golestan national park which located upstream of this dam and covering an area of 92000 ha. It is a living museum with a great diversity of rare species of plants and animals. This park has internationally been recognized as a protected area and each year is visited by large numbers of tourists, (Grwc, 2008).

1.2 *Basin flood features*

The floods of basin are classified two groups, based of occurrence time and topographic characteristics of occurrence place. 1-Spring's flood, which are occurring with long-term duration rainfall and snowmelt in spring season, this flood mostly occur in main river in plain zone and named long reach flood too. 2- Summer floods are occurring within high intensity short-term rainfall and storms, which mostly occur in hill, mountain and plain entrance and have the flash flood characteristics. The floods of 2001 to 2005 were summer and flash flood type the storm centers of these floods were in Golestan's dam basin and just displaced in different years. The biggest and huge flood occurred in 2001 which that's rainfall reached 450 mm in 12 hr, it was the probable maximum precipitation (pmp) based on the past data, and its peak flow reached 3017 cms in Gorgan river in Golestan dam site. It was more than calculated probable maximum flood (pmf) in Golestan dam, see Table 2. The heavy rainfall was main factor in huge flood occurrence; however the geomorphologic of upper catchment and decrease in vegetation coverage such as the cultivation of steep slopes and overgrazing have contributed to quick runoff which caused quicker and larger flood peaks (Jica, 2006).

1.3 *Dams features*

Golestan dam has been constructed in 2000 in the upstream of Voshmgir dam on the main river of Gorgan. Gonbad city with 200000 peoples is located in the 12 km distance of this dam

Table 1. Damages in different floods.

	Year			
Damage	1991	2001	2002	2005
Damages US $ million	18	77	36	69
Killed people	–	247	54	63
Occurance place	Down stream of Golestan dam	Up stream of Golestan dam	Up stream of Golestan dam	Up stream of Golestan dam

Table 2. Hydrologic features of floods.

Year	Rain fall					
	Date	Max intensity mm/hr	Max rainfall mm	Rainfall duration hr	Maximum flow in Golestan dam	Max. Capacity of river in down stream
2001	10/Aug	40	450	11	3017	350
2002	11/Aug	30	108	6	700	350
2005	12/Aug	80	141	5	1500	350
1991	10/May	15	109	48	400	350

in downstream, Figure 1. The main purposes of this dam are irrigation, controlling and reducing sediment to Voshmgir dam. That is homogeneous earth dam with 30 m height, crest length 1300 m, side concrete ogee spillway with wide and capacity 60 m and 1700 cms respectively and calculated pmf is 2389 cms. The irrigating network including 10000 ha located in right bank of dam and in downstream is 4000 ha. The water supply of right bank is done by a pump station, 5.4 cms capacity which installed in reservoir and two bottom outlet pipe, each one 40 cms installed in dam body to release water to downstream irrigation network and Voshmgir dam.

The initial reservoir volume was 86 mcm and regulatable to 101 mcm too. There has not been considered flood control volume. The mean annual sediment of river in dam site is 205 mcm per year (Brekly, 2002)

The Boostan dam is located in upstream of Golestan dam. This dam is homogeneous too. The height of dam is 33, crest length 700 m, side spillway capacity is 700 cms. Annual water flow and sediment in dam site are 45 mcm and 1.7 mcm respectively. The reservoir volume is 48 and regulable to 58 mcm. The Flood control volume equal 18 mcm has been considered. Main purposes of it are irrigation, flood control and reducing sediment to Golestan dam. The irrigation network, 4000 ha is located in right bank and its water supply is done with a pump station installed in downstream of dam, 4.2 cms, on Gorgan river. Two separated outlet pipe with 22 cms installed in different intake level, one mesh or trash rack installed on head of intake which conserve outlet pipe from garbage.

2 FLOOD IMPACT ON DAMS AND PROBLEMS

The Golestan dam has been constructed and completed in 2000 and has been started intake of reservoir in 2001. Up to 2001 flood there has been intaked 10 mcm and the reservoir nearly was empty. The Boostan dam was not constructed up to 2001 flood. Peak flow discharges in floods are presented in Table 3. In 2001 flood maximum flood flow and volume entered to Golestan dam were 3017 cms and 130 mcm respectively, however the calculated pmf and average annual flow volume in Golestan dam site were 2389 and 200 mcm. Peak flood flow entered to Golestan dam was more than designed pmf. Due to such huge flood, the dam was filled quickly by 86 mcm in 8 hrs and were spilled, Figure 2. The maximum overflow discharge on spillway was 230 cms and this discharge routed and reduced in downstream to 150 cms in Gonbad city, Table 3. And there were not felt such huge flood in downstream of Golestan dam in Gonbad city to delta of Gorgan river near Caspian Sea. In the flood of 2002 and 2005, flood flow in Gonbad city were 50 and 30 cms and people in downstream of this dam were not felt flood also. Residences in downstream believed the role of dams in flood control, because in 1991 (which Golestan and Boostan had been not constructed) flood had been occurred, in spring season with peak flow about 400 cms in Gonbad city and were made a lot- of damages in this city to Caspian Sea especially in downstream of Voshmgir dam. the Voshmgir dam was existed that time. If there were not Golestan dam in 2001 flood, the Gonbad city were affected by flood and inundated about 2 m height. In 2002 and 2005 floods this event were occurred too (Lar, 2006).

In 2001 the flood had a lot of damage in upstream of Golestan dam and there have been entered a lot of uprooted trees and branches to reservoir and covered surface of reservoir and entered to outlet pipes too. Further more, about 10 mcm sediment entered to reservoir and then both outlet pipe were closed also the pump station in reservoir became out of work. There were killed more than 254 people in 2001 flood and many of their bodies entered into Golestan dam reservoir and infected the dam water with cholera and made healthy problems.

The flood of 2002 was smaller than 2001 flood and its damages was very low also. (Table 1 and 2). After 2001 and 2002 floods the Boostan dam constructed quickly and finished in first of 2005 (in winter).

Table 3. Flood peak in different stations (m^3/s).

Site	Year				River capacity
	1991	2001	2002	2005	
Boostan dam	257	336	200	891	250
Golestan dam	400	3017	600	900	350
Gonbad city	401	150	50	30	350
Voshmgir cam	419	129	40	20	300
Agh-ghala city	258	20	5	2	100
Caspian sea	273	10	0	1	70

Figure 2. Golestan dam and 2001 flood.

In summer of 2005 another huge flood occurred in Golestan dam, the maximum flood flow into Boostan dam was 900 cms, (Table 2). The dam was empty and in first intake rapidly filled but did not any flow on spillway, flood water volume was about 44 mcm. Furthermore bottom outlet gate has been opened and released to downstream equal 40 cms. In this flood a lot of sediment was entered to Boostan dam and intake establishment in reservoir became out of work up to 50%, damages was in upstream of dams.

During the floods, concerns and challenges of managers of dams and water resources management company in basin especially in 2001 floods was:

1. Dams are delaying type in point view of flood control and there were not ability of flood control operationally and there was not high capacity bottom outlet and other establishment for flood regulation.
2. The river capacity in downstream of dam was low, about 350 cms in Gonbad city and spilled flow more than 350 cms which was hazardous, because of plain topography
3. There was not any experience on reservoir water intake and response of dam body to lowly or rapidly filling of reservoir.
4. Releasing water problem from reservoir. During the flood because all bottom outlet were closed by sediment and big trees.
5. Expectation another flood and damages of over toping on dam body or failure the dam, because there was not, high capacity bottom outlet or fuse gate for emergency discharge, enough capacity of river in downstream and physical conservation in cities and villages.
6. Water intake of reservoir in floods was not according to design water intake, the rate of water intake has been recommended 10 cm/day but the rate was 1 m/hr.
7. Puling out of killed body washed into dam and diseases potential by this reason the dam discharge was necessary but it was not possible because of bottom outlet was closed.

3 MEASURES AFTER FLOODS

After this historical and huge floods, three major measures simultaneously done, 1- Monitoring of installed instruments and rehabilitation of damaged part of dam 2- Revise and modification of Boostan under construction dam 3- Prepare comprehensive flood management plan.

3.1 *Instruments monitoring and rehabilitation*

3.1.1 *Instruments monitoring*

Refer to consideration on dam instruments during and after the flood time no important change was observed to dam behavior. Consideration of installed piezometer at foundation (upstream of cut of wall) showed increasing the pore pressure of 3 piezometer 3.28, 7.28, 5.71 m which means the dam foundation had been saturated on the flood time. All installed downstream piezometers and at foundation, shown no uncommon change. All consideration on inclinometer results showed the maximum deviation less than 2 cm which is an acceptable horizontal deviation. As all settlement devices showed no settles. Then, it can be announced, dam reaction against the flood was normal and acceptable, (Brekly, 2002).

3.2 *Intake opening activities*

After flood slept, it was necessary to reservoir rapid drown down and open the entrance of intake. (because it was completely blocked), therefore all team cooperated for this matter, even a diver team by cheking under water around upper trash rack mentioned that the best way to open and get rid of these mud, trees, … is to use a barge with drug line equipment, to have a grab bucket to be able to remove the mud, … from entrance point.

After hard working of about nonstop 20 days, the top of bottom outlet entrance had been opened and during 30 days reservoir got emptied. After emptying the reservoir, the cleaning all around the intakes, trash rack was started. And then for

improving the situation, 2 steel box of trash rack designed, manufactured, transported and finally installed on top of existing trash rack.

3.3 *Pump station curing*

As well as, the pump station is located in the Dam reservoir, was not out of this flood problem and at the time of flood, and after above activities, still was out of order, but for solving this problem (because so many mud, … had been collected in front of Pump's suction basins). Therefore after long study and, consideration, it was decided to install a steel gate in front of each suction basin to avoid the mud enter to pumps, (Brekly, 2002). The height of gats was 5 m and it means 5 meters of reservoir height was losses.

3.4 *Boostan dam modification*

After the flood, engineering consideration on flood hydrographs at the Boostan Dam location, showed about 22% of peak flood increasing, which means PMF at the Boostan Dam changed from 1317 to 1602 m^3/sec and by the reason of this change, Dam and its related components was modified.

The flood restudy of all existing result reached to increase the Dam Height and spillway elevation 3 meter. And it means the dam reservoir capacity increased about 18 MCM which will be considered for flood control volume, therefore the bottom outlet had been redesign too and the diameter of pipes increased from 1200 mm to 1600 mm which according to this increase, the bottom outlet capacity reached about doubled (44 m^3/sec).

At the end, the intake entrance is designed and constructed in two difference level (upstream) with 5 m of difference level.

3.5 *Comprehensive flood management plan*

Before above floods there was not suitable plan related to flood management. Because, firstly there have not been inspected such huge floods, secondly there were many organization relevant to flood management with different actions did not supported each other. Under such vulnerable situations suffering from flood hazards, effective countermeasures have not carried out and so, to prepare the master plan for disaster mitigation and management of rationalize and integrated various components of structured and non structural measures were necessary, (Jica, 2006). There have been prepared an integrated flood management plan on Gorgan river basin. This flood management master plan covers the entire process of flood occurrence and response and also integrate efforts of the relevant organization. In the other hand, the comprehensive plan including entire processes of flood disaster management such as preparedness, urgent response, recovery, development and prevention measures (Grwc, 2008).

The goal was creation the river basin best management against flood and objectives were: 1- to save people 2- to secure social, environment and economical function. The target of the plan 2020, further more 25 year flood for protection of farmland and village and 100 year flood for protecting structures for road or bridges and town area, also Gorgan river capacity in end of reach near Caspian Sea considered 70 cms as basics of flood plan.

The countermeasures organized into master plan considering area by area and time and all components combined to support each other. The main component of this master plan including: watershed management, river restoration and improvement,

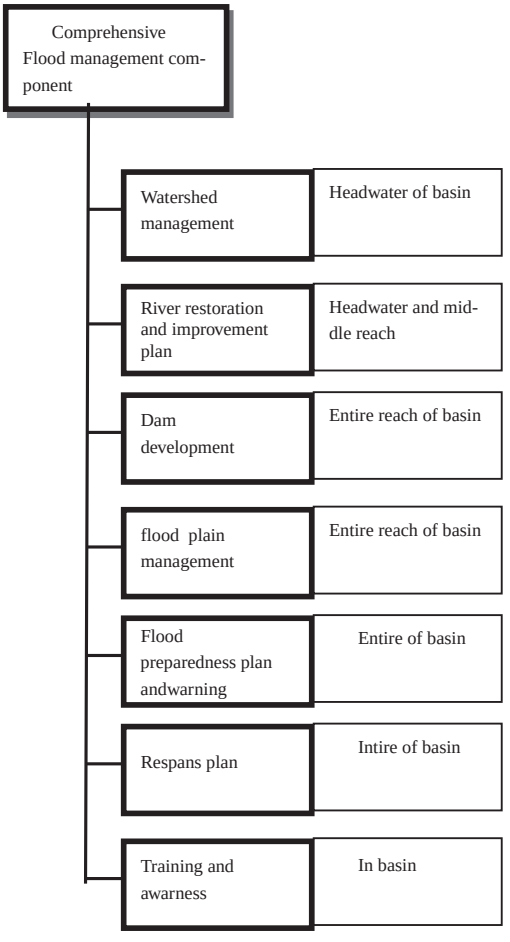

Figure 3. Comprehensive flood management component.

dam development and flood control, flood plan management and hazard map preparation, flood warning and preparedness plan, response plan and training and awareness (Figure 3). There have considered some priority measures mostly non-structural method such as flood warning and flood preparedness focus on community based management, hazard map preparation, training and awareness (Figure 4). The priority plan did or are doing now. For example comprehensive flood warning system in Golestan dam basin has been did and is developing to whole Gorgan river basin.

In sub plan of flood control by dam there have considered some program as follow:

1. To develop new dam and considering comprehensive flood control approaches in related to other measures in dam design.
2. To define the role of existing Boostan, Golestan and Voshmgire dams in flood control and revising exploitation operation plan especially with multi reservoir approaches on Gorgan river.
3. To study failure current dams and hazard map preparation.
4. To prepare flood warning network in each dam basin, preparedness plan or emergency action plan to each dam and considering downstream conservation in each cases of failure dam or water release.
5. Awareness in each step release of water to downstream.

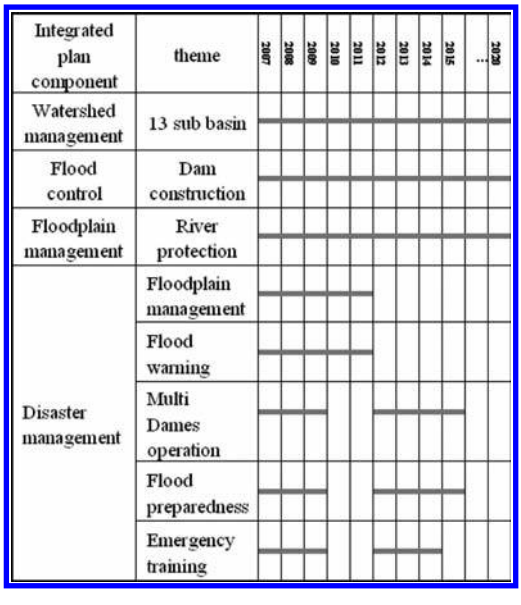

Figure 4. Timing program of master plan.

4 CONCLUSIONS

For the best management of flood river basin there is necessary to prepare comprehensive and integrated flood management plan in whole basin. This plan must cover all related flood mitigation components. In relation flood management by the dam, the comprehensive flood management plan must cover both safety of dam and its downstream. For this purpose, there should consider downstream river capacity and probable damages in design flood and design of dam component concerning to flood such as flood control volume, spillway, …, too. Furthermore to prepare an emergency action plan and warning should be considered in both spillway discharge flow and dam break condition for downstream. Also an emergency instrument monitoring and rehabilitation plan at the time of flood and after that should be prepared. In the highly erosive basin and woodlands to design suitable intake of dam reservoir and its conservation is necessary.

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Rehabilitation of the outlet works of Laing Dam

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ABSTRACT: Laing Dam was completed in 1950 and has been used as a balancing dam and also for domestic water supply to King Williams Town, Bisho and the surrounding towns in the Eastern Cape Province of South Africa. Laing Dam is owned by the Department of Water Affairs and operated by the Amatola Water Board on behalf of the Department. The existing outlet works at Laing Dam could no longer reliably meet the demand from the dam for domestic water supply to the above mentioned surrounding communities. As result it was decided to upgrade the outlet works to ensure a reliable water supply. To ensure continuous water supply during the upgrading to the outlet works, a siphon has been installed during February 2010. This paper describes various civil and mechanical engineering solutions that were designed and implemented during the upgrading of the outlet works.

1 LAING DAM HISTORY

Laing Dam is owned by the Department of Water Affairs and operated by the Amatola Water Board on behalf of the Department. Laing Dam is a 45 m high concrete gravity dam wall with outlet works located on the right flank next to the central uncontrolled ogee spillway section. The dam is shown in Figure 1.

The dam was completed in 1950 and raised in the nineteen seventies by a few metres, including post tensioning of the dam wall. The outlet works can be described as a dry well inlet tower feeding a single 450 mm diameter outlet pipe that bifurcates to the water treatment works and a river outlet (the river outlet is not presently in use). The gate valve near the bifurcation is used to control flow. Each of the six inlets in the dry well has an isolating gate valve to select the level of abstraction in terms of water quality.

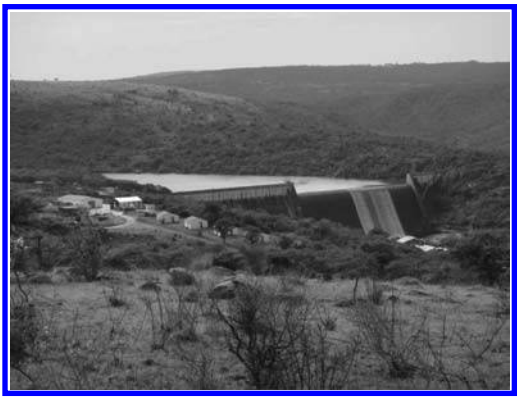

Figure 1. Laing Dam viewed from downstream.

The 2 scour outlets located in the spillway section, each a 600 mm diameter pipe, have a gate valve and both are controlled by a sleeve valve. These river outlets have been refurbished previously and are located in the centre of the spillway section.

The rehabilitation project at Laing Dam includes the total removal of the existing manifold in the inlet tower to be replaced with two supply lines to the water purification plant (this includes removal and replacement of 2 metres of mass concrete underneath the gallery floor to the dry well inlet; and removal of all ladders and platforms (in tower and on left flank) to be replaced with ladders and platforms that comply with the Occupational Health and Safety Act (RSA 1993). Mr J Potgieter is the Mechanical Design Engineer responsible for the design of the mechanical items and Ms H Anderson is the Civil Engineer responsible for the design of the civil items.

Most of the permanent mechanical items have already been designed approximately 10 years ago, however, the project has only been scheduled for implementation during 2009 to 2011 as part of the Department's Dam Safety Rehabilitation Programme. The Dam Rehabilitation Programme has been initiated to address the long outstanding rehabilitation work at government waterworks to comply with the dam safety legislation contained in the regulations read with Chapter 12 of the National Water Act (RSA 1986 and 1998).

2 CONDITIONS

One of the requirements for this project was to perform the rehabilitation work without interruption to the primary water supply to the communities which amounts to approximately 30Ml per day. Mr PB van der Westhuizen, a Senior Mechanical Engineer, designed the siphon as temporary bypass to ensure a continuous water supply. It was agreed with Amatola Water Board that the water supply could be shut down for 5 hours (maximum) to allow for the connection of the siphon to the supply pipe to the water treatment plant of Amatola Water. See Figure 2 showing the water treatment plant immediately downstream of Laing Dam.

The siphon was successfully connected on to the supply pipe during February 2010. Figures 3 to 6 show the installed siphon, some anchor blocks and cutting into the supply pipe line.

Figure 2. Amatola Water Board water treatment works downstream of Laing Dam.

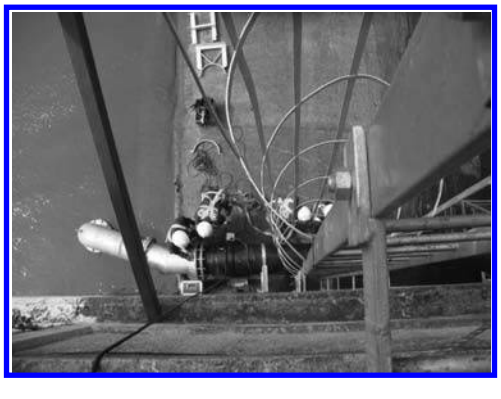

Figure 3. Laing Dam siphon inlet submerged viewed from non-overspill crest with temporary access ladder.

Figure 4. Laing Dam siphon viewed from downstream side of dam wall.

Figure 5. Laing Dam siphon connected onto supply line to water treatment plant.

Figure 6. Cutting into existing supply line to blank off outlet pipe from Laing Dam.

3 LEGISLATIVE REQUIREMENTS

Environmental authorisation was not required for this dam safety rehabilitation project owing to the nature of the work undertaken and subject to compliance with the environmental conditions specified for dam safety rehabilitation projects (RSA 1998).

In order to comply with environmental requirements in general, waste on site is managed by regular removal as well as the construction of an adequate sanitation system for the site establishment. After completion of the project the landscape will be rehabilitated including the removal of other redundant concrete structures.

4 CIVIL CHALLENGES

Katrock a non-explosive demolition agent (expansive material) was selected to be used to crack the area of concrete underneath the gallery floor, after which jack hammers will be used to remove the concrete. Katrock has been tested on site and appear to be an ideal application to use for the removal of the concrete in the dam wall without compromising the structural integrity of the dam wall.

It was also interesting to note that where the Katrock application was tested on reinforced concrete it was found to be less successful, to such an extent that another solution should be sought in cases where reinforced concrete needs to be removed. Luckily at Laing Dam the concrete in the gallery floor area comprises mass concrete only.

5 MECHANICAL CHALLENGES

The confined space inside the outlet tower made it very difficult to distribute the six off-takes to the twin outlet system, while accommodating access ladders in accordance with health and safety regulations and also rendering proper access to all working points.

Owing to confined horizontal space in the gallery area where mass concrete will be removed, the horizontal supply pipes will be installed vertically above each other to overcome this limitation. This will require the removal of at least 2 metres of mass concrete below the gallery floor area.

Temporary isolation of the six gooseneck pipe intakes was done by divers installing a plug on each intake. Fortunately the condition of the gooseneck intakes was found to be in relatively good condition resulting in drip tight sealing of all six intakes.

Another challenge included the planning, design for installation and commissioning of the siphon within the allowed 5 hour time span.

The concrete top structure of the outlet works tower cannot support the crane loads and a mechanical structure had to be designed and built to support the crane inside the tower top structure. This configuration will allow the removal of existing pipes and valves.

6 CONCLUDING REMARKS

Laing Dam rehabilitation project is in the early stages of implementation with a total project timeframe of approximately eighteen months and a budget of €423 000.

Civil and mechanical engineers have collaborated successfully up to date to ensure that the project remains on target, within specifications and budget. It is apparent that the work done by the engineers in this Department is complemented by a competent contractor and also including the good work done by the diving contractor. Continued efforts will be made by this team to ensure a successful completion of the project.

These rehabilitation projects should receive timeous financial support to extend the life of the bulk water infrastructure assets in order to provide a safe and sustainable bulk water supply to communities.

ACKNOWLEDGEMENTS

The authors wish to express their gratitude to Messrs H Krynauw and P Duminy making the photographs on this project available and Mr PB van der Westhuizen for his valuable contributions on this project to date. The contractor Spec (Pty) Ltd is also thanked for their role and collaboration on this project. The Department of Water Affairs is thanked for permission to publish this paper, however, the opinions expressed are however those of the authors and do not necessarily reflect the views of the Department.

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La gestión de avenidas y el cumplimento de las normas de explotación. Aplicación en los Servicios 1 y 5 de la Confederación Hidrográfica del Ebro

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RESUMEN: Las Normas de Explotación de las presas pertenecientes a la Confederación Hidrográfica del Ebro establecen en el apartado de Explotación en Situaciones Extraordinarias, y más concretamente en el de Situaciones en Avenida, un procedimiento de actuación entre los que se encuentran la generación de una documentación de declaración de situaciones de avenida, así como la cumplimentación de un Estadillo de Control de Avenidas. Con el fin de dar cumplimiento a las normas de explotación en situación de avenida se ha desarrollado una herramienta informática para las presas de los servicios n° 1 y n° 5 de la Confederación Hidrográfica del Ebro, que automatiza las operaciones a realizar, y permite analizar situaciones de avenida históricas y escenarios probables, comprobando la evolución de los niveles y la superación o no de los distintos umbrales de aviso, generando la documentación correspondiente.

ABSTRACT: The Safety Regulations within the operation of the Ebro's Hidrological Confederation dams, and more in particular within the operation during floods, settle a procedure to indicate the management for minimizing the risk involved with such events, mainly consisting in the generation of public notifications to the subjects involved and the filling out of a tool for the operation of the dam in the flood control. In order to optimize this management, a new computer tool has been developed for Services n° 1 and n° 5 of the Ebro's Hidrological Confederation, for the automation of the procedure, allowing as well to analyze historical floods, forecasting of scenarios, and generating all the public notifications when necessary instantly.

1 INTRODUCCIÓN

1.1 *Las Normas de Explotación en la gestión de avenidas*

La obligatoriedad de disponer de Normas de Explotación estaba ya recogida en la Instrucción para el Proyecto, Construcción y Explotación de Grandes Presas (1967), en su Artículo 94, bajo la denominación de "Normas de explotación, conservación y vigilancia".

En el "Reglamento Técnico sobre seguridad de presas y embalses" (1996), se exige (Articulo 5.7) la existencia de unas Normas de Explotación, respecto a las que se establecen una serie de requisitos a tener en cuenta, entre los que se encuentran los derivados de las situaciones extraordinarias de avenidas, con el fin de garantizar la seguridad de la presa y el embalse.

En base a estas premisas, la Confederación Hidrográfica del Ebro, establece en los apéndices 16 y 17 de las Normas de Explotación de sus presas, un procedimiento de actuación en situaciones de avenida cuyo contenido se refleja en un cuadro general de definiciones y de actuaciones en situación de avenida, y se ejecuta en unos partes de declaración y notificación de situaciones de avenida y en un impreso tipo y cuadro para la gestión de las mismas mediante la operación de sus órganos de desagüe. Los partes de declaración existentes son tres:

- Notificación por fax de declaración y cambio de situación de avenida (FAV1).
- Registro de comunicación telefónica de vertido de caudales en situación de avenida (FAV2).
- Informe de incidencias debido a situaciones de avenida (FAV3).

De este modo, se definen las duraciones y objetivos a cumplir de cuatro situaciones de; aviso, avenida, daños y normalización, en función de la superación de unos niveles de resguardo, caudales entrantes o caudales de daños aguas abajo de la presa, repartiendo las funciones y actuaciones a ejecutar para cada una de ellas, de manera que en ningún momento se vea comprometida la seguridad de la presa.

Las herramientas de las que se dispone hoy en día en los organismos de cuenca, implementadas en los Sistemas de Ayuda a la Decisión, mediante el conocimiento de predicciones meteorológicas avanzadas y un Sistema Automático de Información Hidrológica moderno y completo, facilitan la gestión conjunta de las distintas presas en situaciones de lluvia extrema.

Sin embargo, ya sea bien porque existe la remota posibilidad de fallo de dichas herramientas, bien porque existen presas de menor entidad cuya gestión se sale de la gestión global de la cuenca por motivos de optimización de los recursos, debe de existir una herramienta automatizada e individualizada para cada presa que permita cumplir los requisitos anteriores de seguridad.

El impreso tipo que define la operación de los órganos de desagüe para la gestión de las avenidas, se trata de un estadillo regido por un conjunto de tablas, que indica la operación de manera mecánica. Sin embargo, en las situaciones de stress en las que es de aplicación, su utilización se hace difícil, rígida y pesada, aumentando las probabilidades de errar, así como dificultando la predicción y la detección de la superación de los distintos umbrales que llevan a la generación de los partes de aviso y posterior notificación a los distintos afectados.

1.2 *Los Servicios de Explotación n° 1 y n° 5 en la Cuenca del Ebro*

Los Servicios de Explotación n° 1 y n° 5 de la Cuenca del Ebro se enmarcan en la cabecera y margen derecha del curso alto del río Ebro, y bajo su gestión se encuentran cinco grandes presas, como son la presa del Ebro (Cantabria), de 540.0 hm^3, la presa del Val (Zaragoza) de 25.0 hm^3 en el Servicio n° 1 y las presas de Mansilla, González Lacasa y Pajares (La Rioja) de 75.0, 32.9 y 35.0 hm^3 respectivamente en el Servicio n° 5.

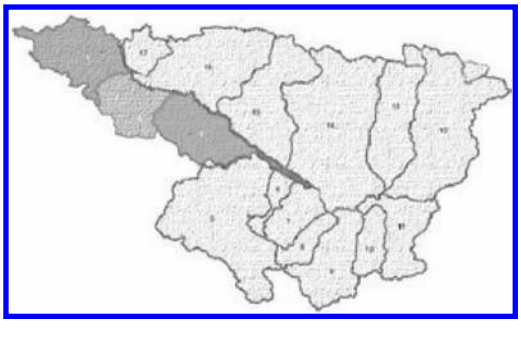

Figura 1. Situación geográfica de los Servicios de Explotación nº 1 y nº 5 de la Confederación Hidrográfica del Ebro dentro de la Cuenca del Ebro.

2 OBJETIVOS Y METODOLOGÍA

2.1 *El Método de Dordogne*

Las normas de explotación de las presas de la cuenca del Ebro utilizan el método Dordogne propuesto por la Guía francesa, en su versión original como método de gestión de las avenidas.

Este método calcula el caudal que debe verterse en cada momento en función del nivel del embalse y de su velocidad de subida, en la hipótesis de que el personal de explotación se encuentre aislado y sólo puede medir la cota de agua en el embalse. Su objetivo es evitar la superación del nivel de vertido libre, para lo cual estima el incremento que debe aplicar al caudal que se está vertiendo de forma que, en la hipótesis de que el caudal entrante se mantuviera constante, se reserve un volumen de embalse bajo el de vertido libre que permita almacenar la diferencia entre el caudal entrante y saliente si éste crece linealmente hasta igualar a aquél en el momento de alcanzar el nivel de vertido libre. Con estas hipótesis, el incremento de caudal a aplicar al vertido en cada instante es el siguiente:

$$\Delta S = \left(\frac{A \cdot \Delta H^2}{2 \cdot \Delta T \cdot (V_L - H)} \right) \tag{1}$$

siendo A la superficie del embalse en m^2 a cota H, ΔH la subida de nivel desde la última lectura, V_L el nivel de vertido libre y ΔT el intervalo de tiempo entre lecturas. El método de Dordogne no especifica la forma de operación cuando el embalse baja.

La metodología expuesta se simplifica en un organigrama que indica la casuística de las operaciones a realizar, en función de que el embalse o el hidrograma estén subiendo o bajando y de la posición del nivel de agua.

En definitiva, un total de trece casos, que sin embargo se gestionan mediante cinco operaciones distintas que se repiten en según que casos y que facilitan la implementación del método en una herramienta informática que lo automatice. De estas cinco operaciones, una de ellas se trata del método de Dordogne propiamente dicho, correspondiente a dos casos (gradiente del caudal de entrada positivo y negativo) en los que el nivel de embalse

sube y además se encuentra por encima de la curva de resguardo de T años y por debajo del Nivel Máximo Normal del embalse.

3 ESTADILLO DE GESTIÓN DE AVENIDAS

3.1 *Introducción*

En el proceso de gestión de las presas en situación de avenida es necesario analizar cómo manejar y realizar cálculos con la gran cantidad de datos existente, de manera que ayuden a la toma de decisiones en cuanto a la operación de los desagües de la presa de manera rápida y eficaz.

Esta herramienta no debe considerarse como un elemento aislado, ya que el juicio crítico en función de la experiencia y conocimientos reales de los diferentes episodios acaecidos es un factor predominante, de tal forma que la aplicación informática que se presenta se integre como una herramienta más para el personal técnico cualificado que vela por el buen funcionamiento de las distintas presas.

3.2 *Aplicación informática*

La gran cantidad de información que se debe de manejar durante un evento extraordinario de avenida, requiere que el estadillo automatizado de gestión de avenidas aglutine toda la información de manera ordenada y dinámica, para facilitar su consulta y ayuda a la toma de decisiones. Por ello, junto a la interfaz de introducción de los datos de entrada se presentan diferentes menús que contienen datos sobre:

- Características y planos de la presa.
- Hidrogramas teóricos.
- Niveles de explotación.
- Curvas de vigilancia.
- Teléfonos y direcciones de interés.
- Curvas de vigilancia.
- Caudales de daños aguas abajo y características de las afecciones.
- Formularios de notificación.

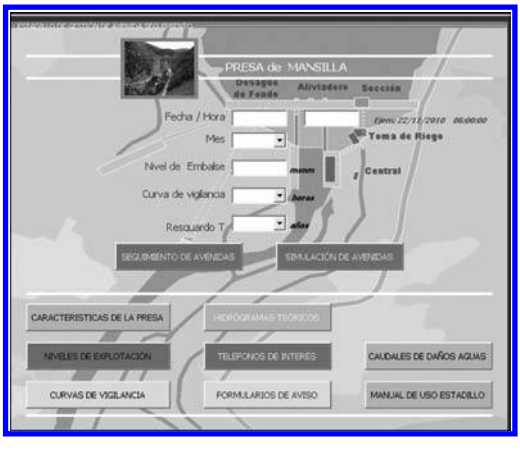

Figura 2. Interfaz de datos de entrada y gestión y consulta de la información de la presa.

Mediante la introducción de las lecturas de embalse detectadas durante el evento extremo, la aplicación calcula directamente el caudal que se debe de desaguar según la metodología expuesta (Método de Dordogne), en función del gradiente del nivel del embalse y del hidrograma de entrada calculado.

La automatización del impreso tipo en un estadillo de gestión de avenidas, permite operar los distintos órganos de desagüe de la presa, obteniendo directamente el caudal desaguado en función de la apertura de compuertas y del nivel del embalse, con la implementación de las curvas de gasto de cada uno de ellos.

De esta manera, permite adaptarse o anticiparse a los caudales de salida que se indican según el método de Dordogne.

En cuanto a la generación de partes de declaración y notificaciones, la aplicación permite generarlos bien automática o manualmente. Además se han introducido otros partes derivados de la gestión ordinaria de la presa y de la entrada en el escenario 0 del Plan de Emergencia en situación de avenida, que pueden estar vinculados al inicio y final del evento extraordinario. Estos partes de declaración son:

- Notificación oral/fax declaración cambio de escenario (F101/F102).
- Comunicación telefónica de vertido de caudales en situación normal (FC1).

Automáticamente se revisan las columnas de nivel de embalse y de caudal total saliente y se verifica si se superan los umbrales que definen cada una de las situaciones de aviso:

- Nivel de Embalse >Curva de vigilancia V. Situación de Aviso (FAV 1 y FAV2).
- Nivel de Embalse > Resguardo para T años. Situación de Avenida (FAV 1, FAV 2 y FAV 3).

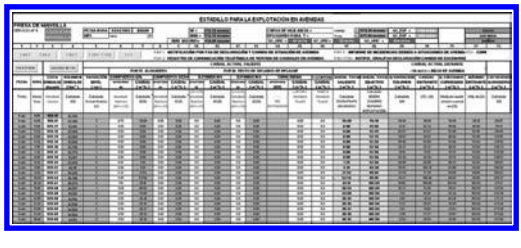

Figura 3. Estadillo automatizado de gestión de avenidas.

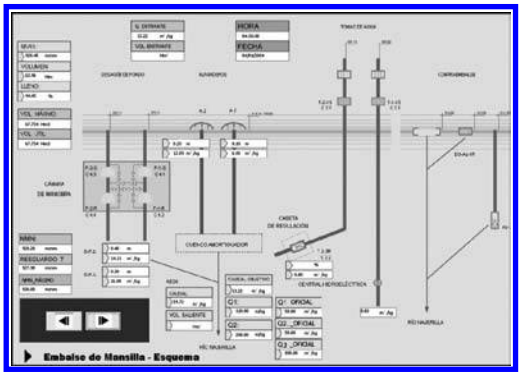

Figura 4. Diagrama de datos de salida.

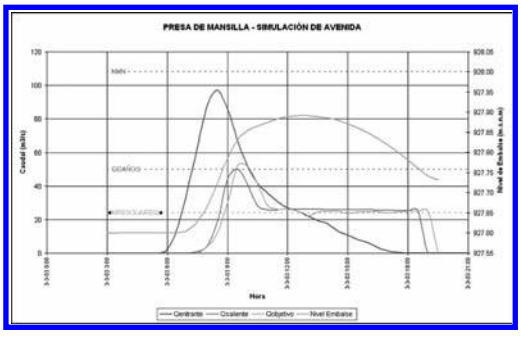

Figura 5. Gestión de una avenida teórica realizada para la presa de Mansilla.

– Caudal total saliente > Q1 (desbordamiento del cauce natural). Situación de Avenida (FAV 1, FAV 2 y FAV 3).
– Caudal total saliente > Q2 (afección a viviendas aisladas, zona agrícola importante e infraestructuras secundarias). Situación de Daños (FAV 1, FAV 2 y FAV 3).

Para una mejor visualización y comprensión de la gestión realizada, la aplicación informática implementa una interfaz gráfica con el diagrama de la presa y los distintos datos de salida para cada uno de los órganos de desagüe y de la presa en general.

Finalmente la aplicación se complementa con una versión de simulación de avenidas, que permite ensayar distintas operaciones de embalse mediante la introducción de un hidrograma de entrada (teórico o real), y posibilita la recreación de una gestión de avenida con la posibilidad de seguir la gestión según el método de Dordogne expuesto.

4 CONCLUSIONES

La herramienta informática presentada flexibiliza la gestión de las avenidas y facilita el cumplimiento del protocolo de actuaciones establecido en las Normas de Explotación de la presas.

Las personas responsables de la gestión de la presa obtienen formación e información que permite optimizar su trabajo desde el punto de vista de la gestión de avenidas, ya que amplía el conocimiento del comportamiento de las presas, lo cual redunda en una previsión de las operaciones a realizar en los órganos de desagüe, resguardos a establecer en diferentes hipótesis, etc.

Permite recalcular de manera fácil y directa los resguardos asociados a los distintos periodos de retorno ante eventuales cambios en la situación de los órganos de desagüe o en los valores hidrológicos esperados.

La gestión automatizada de las labores de gestión de explotación de la presa en situación extraordinaria de avenidas, es una herramienta necesaria para el Ingeniero Encargado de la Presa.

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Economical aspects water resources management and appropriate pattern for agricultural water demand management in Iran

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ABSTRACT: From hydrological point of view, Iran is divided into 6 main drainage basins and potential harvestable surface water resources are estimate at 117 bln. m^3. Economically and technically usable surface water amounts annually to 86 bln. m^3. Nearly 50.5 bln. m^3 of water obtained from the precipitation in high mountainous region along with precipitation in plains is used for recharge of aquifers. Due to the drastic growing of water consumption the variation of ground water balance indicate obviously a rapid decreasing of 5 up to 6 bln. m^3 and the surface flow infiltration has declined from 13.2 bln. m^3 per year.

In the agricultural sectors the water allocation and supply is arranged by the regional sectors of ministry of energy, but it agriculture sector leads management on demand, consumption optimization and on-farm. In addition, an autonomous organization as sponsor and supporter observers and supervise the environmental issues.

With respect to long term water resources development, supreme water council with consumer sectors participation performs demand and supply conjoint management, which required funds, will be provided through general financial resources, leading to establish of hydraulic structures such as dams, ponds, irrigation networks, etc. and the ministry of energy and ministry of Jihad-H Agriculture are accountable for performance of supreme water council policies.

In Iran was not accomplished any study on agricultural water demand, so agricultural water demand is estimated consider to irrigation land farm and water efficiency in this investigation.

Due to international challenges, facing the limitations of new fresh water supply, the government urged highly the optimization and productivity enhancement this substantial vital stuff.

Regarding to long term development strategies of water resources, water supreme council with cooperation of public consumer foundations relating to coincides supply and demand management in water supply put into effect, which finally required founds should be provide through general (financial) resources to perform divers hydraulic structures whereas ministry of energy is.

1 INTRODUCTION

Producing more food without more water will not be easy, but some new approaches will help. Managing water to meet future needs will also involve making water use more efficient, particularly in agriculture, which accounts for 70 percent of freshwater withdrawals from rivers and groundwater (figure 3.2).

There appears to be scope for increasing the productivity of water in rainfed agriculture, which provides livelihoods for the majority of the world's poor, generates more than half of the gross value of the world's crops, and accounts for 80 percent of the world's crop water use (Molden, 2007).

Options described in the next section, include mulching, conservation tillage, and similar techniques that retain water in the soil so that less is lost to evaporation and more is available to plants. Other options involve small- scale rainwater storage, sometimes called water harvesting.

Of the various interventions to increase rainfed production, some (mulching and conservation tillage) divert some water that would otherwise evaporate unproductively. Others (water harvesting, groundwater pumps) divert some water that would otherwise have been available to users downstream. When water is plentiful, impacts on other users are imperceptible, but as water

becomes scarcer, the impacts become more important. Once again, comprehensive accounting for water and integrated planning of land and water at local, watershed, and regional scales can make these interventions productive, by ensuring that the tradeoffs are properly evaluated.

Irrigated agriculture is expected to produce a greater share of the world's food in the future, as it is more resilient to climate change in all but the most water-scarce basins (Molden, 2007).

Crop productivity per hectare will have to increase, because there is little scope for increasing the total area under irrigation. Indeed, irrigated land is expected to increase by just 9% between 2000 and 2050 (Rosegrant, Cai and Cline 2002). Water productivity (in this case, agricultural output per unit of water allocated to irrigation) will also have to improve, given the increasing water demands of cities, industries and hydropower. New technologies have the potential to increase water productivity when combined with strong policies and institutions (Indian Financial Express on December 1, 2008).

Getting more "crop per drop" involves a complex combination of investments and institutional changes. Countries from Armenia to Zambia are investing in new infrastructure that delivers the water efficiently from the reservoir to the crops, reducing evaporative losses. However, as the example of the Moroccan farmers described earlier indicates, the investments can work only if local institutions deliver the water reliably, farmers have a voice in decision making, and they can get the advice they need on how to make the most of the new infrastructure or technological developments. New infrastructure will help water management only if combined with strong quantitative limits on each individual's water consumption, covering both ground and surface water. Otherwise, the increased profitability of irrigation will tempt farmers to expand their cultivated area or double- or triple- crop their fields, drawing ever more water from their wells. This is good for the individual farmer, certainly, but not for the other water users in the basin (De Fraiture and Perry, 2007). Good crop management can increase water productivity by developing varieties resistant to cold so that crops can be grown in the winter, when less water is required (Perry, Steduto, Allen and Burt).

Growing crops in greenhouses or under shade screens also can reduce the evaporative demand of open fields, though it does increase production costs (Moller, Tanny, Li and Cohen, 2004).

When crops die before they produce their yields, the water they have consumed is wasted. Therefore more widespread adoption of drought- and heat-tolerant varieties will increase water as well as agricultural productivity (Perry, Steduto, Allen and Burt). Well- timed applications of irrigation water can also help. If farmers do not know exactly how much water is needed, they often over irrigate because a little extra water is less harmful to yields than too little water. By monitoring water intake and growth throughout the growing season, farmers can deliver the exact amount of water that their crops need and irrigate only when really necessary. Remote- sensing systems are beginning to allow farmers to see the water needs of plants with great accuracy even before the plants show signs of stress (www.fieldbook.com).

But because of the technological requirements, precision agriculture of this type is limited to a small number of the world's farmers (Perry, Steduto, Allen and Burt). Climate change will depress agricultural yields. Climate change will affect agriculture directly through higher temperatures, greater crop water demand, more variable rainfall, and extreme climate events such as foods and droughts. It will increase yields in some countries but lower them in most of the developing world, reducing global average yields (Müller, Bondeau, Popp, Waha and Fader, 2009). In mid to high latitudes, local increases in temperature of only 1–3°C, along with associated carbon fertilization (WDR, 2010) and rainfall changes, may have small beneficial impacts on crop yields (Easterling, Aggarwal, Batima and et al., 2007). Kazakhstan, the Russian Federation, and Ukraine are all geographically positioned to benefit from these temperature increases, but they may not be able to capitalize fully on the opportunities. Since the breakup of the Soviet Union, together they have removed 23 mln. ha of arable land from production, almost 90% of which was used for grain production. Although world grain yields have been rising on average by about 1.5% a year since 1991, yields in Kazakhstan and Ukraine have fallen, and Russia's yields risen only slightly. If these countries are to take advantage of the warming temperatures to increase agricultural production, they will have to build stronger institutions and better infrastructure (Fay, Block and Ebinger, 2010). In most developing countries, climate change is projected to have an adverse effect on current agriculture. In low- latitude regions even moderate temperature increases of another 1–2°C will reduce yields of major cereals. One assessment of multiple studies estimates that by the 2080s world agricultural productivity will decline 3% under a high- carbon-emission scenario with carbon fertilization or 16% without it (Cline, 2007). For the developing world, the decline is projected to be even larger, with a 9% decline with carbon fertilization, and 21% without.

Efforts to mitigate climate change will put more pressure on land. In addition to reducing yields, climate change will put pressure on farmers and other land managers to reduce greenhouse gas

emissions. In 2004 about 14% of global greenhouse gas emissions came from agricultural practices. This includes nitrous oxide from fertilizers; methane from livestock, rice production, and manure storage; and carbon dioxide (CO_2) from burning biomass, but excludes CO_2 emissions from soil management practices, savannah burning, and deforestation (IPCC (Intergovernmental Panel on Climate Change) 2007a).

Developing regions produce the largest share of these greenhouse gas emissions, with Asia, Africa and Latin America accounting for 80% of the total. Growing populations, more carnivorous palates, and climate change will require large increases in agricultural productivity. The amount of land needed to feed the world in 2050 will depend significantly on how much meat people eat. Meat is a resource- intensive way for humans to consume protein, because it requires land for pasture and grain feed.

The resource implications vary with the type of meat and how it is produced. Producing 1 kg of beef can take as much as 15,000 liters of water if it is produced in industrial feedlots in the United States (Beckett and Oltjen, 1993), (Hoekstra and Chapagain, 2007).

But extensive beef production in Africa requires only 146–300 liters per kg depending on the weather (Peden, Tadesse and Mammo, 2004). Per kg beef production is also greenhouse-gas intensive, even compared with other meat production, emitting 16 kg's of CO_2 equivalent (CO_2) for every kg of meat produced (Williams, Audsley and Sandars, 2006).

2 ISLAMIC REPUBLIC OF IRAN

2.1 *Geography*

Iran with the territory of 1.648.195 km^2 is bordered to the north by the states of Armenia, Azerbaijan and Turkmenistan (Republics of the former USSR) and the Caspian Sea; to the east by Afghanistan and Pakistan; to the south by the Gulf of Oman and the Persian Gulf; and to the west by Iraq and Turkey. Iran is about one-fifth the size of the USA and more than three times larger than France. The country is dominated by three mountain ranges: the fertile, volcanic Sabalan and Talesh ranges in the north-west; the very long, Jurassic-era Zagros range, down the western border; and the dominant Alborz range, home of Iran's highest mountain, the permanently snowcapped Damavand (5670 m), to the north of Tehran. The two great Iranian deserts the Dashti Kavir (more than 200.000 km^2 and the Dashti Lut (more than 166.000 km^2) occupy most of the northeast and east of the central plain. About 14% of Iran is arable land; 8% is forest; 55% is non-arable pastures; and the remaining 23% is desert. About 52% of the country area is mountainous and deserts and some of 16% of the country has

an elevation of more than 2000 m above sea level. The cultivated area is estimated about 51 mln. ha or is 31% of the total area of the Iran. In the past three years it was considered 18.5 mln. ha.

2.2 *Climate*

Because of its size, variety of topography and altitude, Iran experiences great extremes of climate. Winters (December to February) can be unpleasantly cold in most parts of the country; while in summer (June to August) temperatures go up as high as 40°C. The average annual temperature increases from northwest to southeast and varies from minus 10°C in Azerbaijan to 25–30°C in the south and southeast the country. The annual precipitation increases with elevation, and spatial and seasonal variation is high. Iran's annual rainfall is 250 mm which varies from 50 mm in the desert to more than 2000 mm in Caspian Sea region. Country can be divided into eight major climatic zones ranging from arid in the central plain and along the southern cost to semi-arid Mediterranean along parts of the western and northern portion of the country to very humid on the southern shore of Caspian Sea. Precipitation ranges from 100 mm to 550 mm and due to high evatranspiration rates during spring and summer. Total annual precipitation is 413 bln. m^3.

2.3 *Water resources*

From Hydrological point of view, Iran is divided into 6 main drainage basins and potential harvestable surface water resources are estimate at 117 bln. m^3. Economically and technically usable surface water amounts annually to 86 bln. m^3. Spatial variation of water resources is also very large. The largest river in Iran is Karoon in Khuzestan province with 24 bln. m^3 annual volume. Seasonal and annual river flow regimes are also very irregular and safe yield from surface water necessitates the construction of storage facilities. More than 13.000 water supply project including earth dams, diversion dams, pump station and etc put in to operation as of the end of 2003 and total harvested water is equal to 7.2 bln. m^3.

The volume of water obtained from the precipitation is about 413 bln. m^3 in the whole land of which 295 bln m^3. Approximately 71% of the total rainfall returns to the atmosphere by evatranspiration. Almost 105 bln. m^3 of the remaining water along with 12 bln. m^3 entering from borders from the surface water resources of Iran. Nearly 50.5 bln. m^3 of water obtained from the precipitation in high mountainous region along with precipitation in plains is used for recharge of aquifers. Due to the drastic growing of water consumption the variation of ground water balance indicate obviously a rapid decreasing of 5 up to 6 bln. m^3 and the surface flow

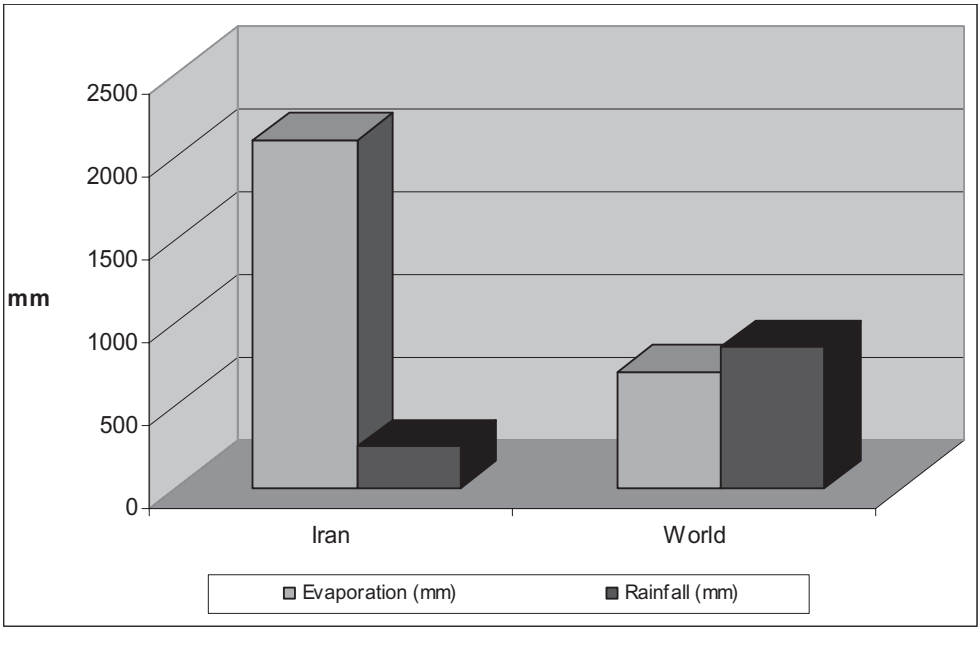

Figure 1. Evaporation and rainfall means in the world and Iran.

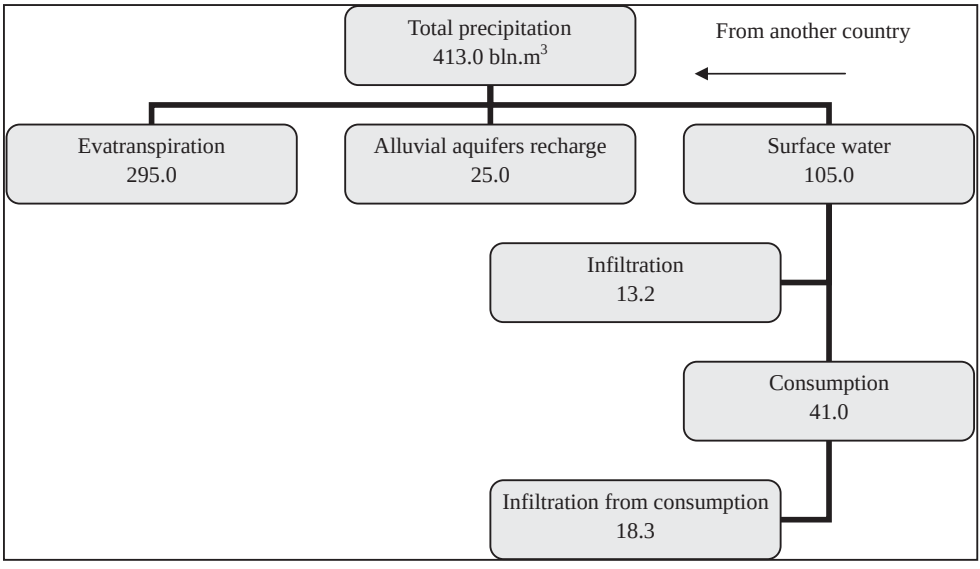

Figure 2. Contribution of the precipitation on the water balance of Iran.

Table 1. Water resources utilization of the Iran by different consumptions.

Resources	Utilities	Agriculture, bln. m^3	Drinkable, bln. m^3	Industry mine, bln. m^3	Summary, bln. m^3
Surface flow	*A*	23.3	2.0	0.4	25.7
	B	11.9	–	–	11.9
	Total	35.2	2.0	0.4	37.6
Underground	*C*	40.3	3.0	0.7	44.0
	Springs	6.5	0.5	–	7.0
	Qantas	4.0	0.5	–	4.5
	Total	50.8	4	0/7	55.5
Sum – Total		86	6	1.1	93.1

Notes: Data from Ministry of Energy (2001).
A- Reservoir and diversion pumps; B- Traditional utilization; C- Deep and semi deep wells.

infiltration has declined from 13.2 bln. m^3 per year. These changes in water supply and demand in the whole land caused many serious problems, which are reverted to the swift population growth in the two last decades. Due to all these constraints and obstacles, the major step in this thematic were the small and large dams to construct as the assured solution considering infrastructure measurements.

2.4 *Agriculture*

Because of its size, variety of topography and altitude Iran has great extremes of climate and therefore the type of agricultural products depends on the climatic condition of each area. In the south of Iran with its typically hot weather condition, the main product is date with a world-famous quality. One of the Iranian export products is fruit, which is on high demand in the neighboring countries, like Russia in the north and also in Europe. From 163.6 mln. ha of the country's total land area only 11% is cultivable and 63% have not been used, and given the fact that 18.5 million ha of the present farms are being used with 50% to 60% capacity; about one-seventh of the total cultivated land is sufficient to meet domestic demand. Pistachio is one of the agricultural products that is associated with Iran in the world economy. The other important agricultural product is Saffron, which is beautiful, delicious, and expensive and the best in the world. Khorasan Province in the Northeast of the country is the biggest producer of Saffron and in

Figure 3. A tea plantation in Lahijan of Iran.

Figure 4. A rice field in North of Iran.

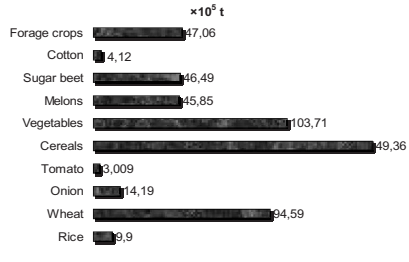

Figure 5. Production of agricultural foods in Iran (amounts are related to 2001).

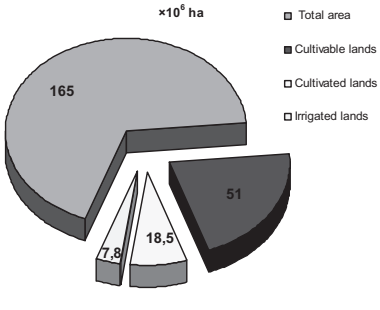

Figure 6. Cultivation soil resources of Iran.

1999–2000 years it exported 69 t at the value of $27.800.000.

3 ESTIMATION OF WATER DEMAND FOR AGRICULTURE OF IRAN

In the agricultural sectors the water allocation and supply is arranged by the regional sectors of ministry of energy, but it agriculture sector leads management on demand, consumption optimization and on-farm. In addition, an autonomous organization as sponsor and supporter observers and supervise the environmental issues. This organization implements several important programs in the country, which play a vital role in the protection of eco-systems. One of the most remarkable measures by the government is a great campaign against observing the equity of distribution and fair management in different sectors. Other activities include operating of water management if forming an inquiry commission for water management policy, holding seminars, and conferring environmental marks and outstanding water saving devices. The general laws policy of the country, which substantially based on food security supply, is really economically not justifiable, due to a huge investment and expenses. This terminology specified the volume delivery of water, required in specific time and place in order to requirement of water for the crop in determined region, regarding to the national act or document approved in 1996 confirmed by council of ministers, agricultural consumption optimization pattern.

Out of 18.5 million ha of cultivated lands, 7.8 million ha are classified as Irrigation has been practiced in Iran for centuries. So called "traditional schemes "cover 6.0 mln. ha. Of which 4 million ha are supplied by groundwater (wells, springs, ghanats) and the remaining 2 irrigated lands within the classified irrigated area covers 1.3 million ha are irrigated by run—off—river diversions. Traditional schemes are equipped with

Table 2. Iran hydrological water balance.

mm	bln. m^3						
P	Q	E	R	B	V	W	N
250	413	296	117	13	130	105	25

Notes: P- Annual precipitation's mean of the country, Q- Yearly precipitation volume, E- Yearly Eva trans privation reduction, R- Yearly remain, B- Yearly brooders outbound, V- Volume of renewal water resources, W- Volume of surface water, N- Natural recharge of precipitations.
Source: Ministry of energy (2001).

Table 3. Cultivation area and productivity of agricultural kinds in Iran.

Kind	Cultivate area (ha)	Productivity (t)
Wheat	2177901	6651567
Barely	622923	1615682
Paddy	514791	1990223
Grain maize	172721	1064186
Cereals	154195	205142
Cotton	186640	399244
Sugar beet	171658	4649017
Oily seed	130147	168759
Sugar cane	36860	3195333
Tobacco	12304	13337
Green vegetables	417855	9998299
Patches	252350	4458448
Fodder	619708	6498437
Other agra. products	53834	103991
Total	5523887	41011665

Notes: Data from Agricultural Jihad Ministry (2000).

Table 4. Cultivation area and productivity of watered orchards in Iran.

Kind of products grained fruits	Cultivate area (ha)	Production (t)
Apple – pear – quince	196656	2351881
Cherry – tomato – peach – plum	143951	1128420
Pint site fruits grape – mulberry strawberry	230961	2332192
Mealy fruits nut – almond hazelnut pistachio	589351	370908
Micro thermal fruits – meddler – burberry – hawthorn	8771	12825
Tropical fruits – composite – date pomegranate – fig – tea	559466	5187934
Tropical fruits – banana – mango	5158	56286
Other orchards saffron	139878	118605
Total	11774988	1874192

Source: Agricultural Jihad Ministry of Iran (2000).

unlined canals and density of canals in command area is generally very low. Government involvement in modern irrigation development started in the 1950s and the total equipped area has reached 1.5 million ha in 1995, of which 1.1 million ha are supplied by groundwater and 200000 ha are fed by run—off—river pumps.

According to the laws of management and planning organization (MPO), each supply, distribution and conveyance project, suggested through relevant system (organization) proposed for feasibility studies and performance will be considered and ratified by MPO in the annual country general main budget. After finance and accomplishment of phases 1 and 2 regard to the distinguished user rate (30%) and the remain (70%) will assumed by the government. in case of any further (extra) performance and aftermath costs in the phase 3 it is allowed to increase the expenses up to 25% which will be paid by the MPO in case of further excess of costs, the M.P.O will assume in order to laws performances adjustment up to 25% additional expenses.

In Iran was not accomplished any study on agricultural water demand, so agricultural water demand is estimated consider to irrigation land farm and water efficiency in this investigation (Table. …)

According Hoekstra, A.Y. and Chapagain, A.K. (2008) the water footprint of a nation refers to the total amount of water that is used to produce the goods and services consumed by the inhabitants of the nation. The total water footprint of a country includes two components: the part of the footprint that falls inside the country (internal water footprint) and the part of the footprint that presses on other countries in the world (external water footprint). Water footprint of a nation—Is defined as the total amount of water that is used to produce the goods and services consumed by the inhabitants of the nation. The national water footprint can be assessed in two ways. The bottom-up approach is to consider the sum of all goods and services consumed multiplied with their respective virtual-water content. In the top-down approach, the water footprint of a nation is calculated as the total use of domestic water resources plus the gross virtual-water import minus the gross virtual-water export. Internal and external water footprint—The total water footprint of a country includes two components: the part of the footprint that falls inside the country (internal water footprint) and the part of the footprint that presses on other countries in the world (external water footprint). The distinction refers to the appropriation of domestic water resources versus the appropriation of foreign water resources. Water footprint of a product—The water footprint of a product (a commodity, good or service) is the volume of freshwater used to produce the product, measured at the place where the product was actually produced. It refers to the sum of the water use in the various steps of the production chain. The 'water footprint' of a product is the same as its "virtual water content" (Hoekstra and Chapagain 2008).

In the Table … present water footprint of the Iran on comparison to same countries of former Soviet countries.

Table 5. Water mutilation of different resources for cultivated area and irrigation reticence.

Water supply resources		Volume of utilizations (bln. m^3)	Cultivated area (10^3 ha)	Efficiency (%)
Surface flow	*A*	23.3	1600	32
	B	11.9	900	35
	Total	33	2500	33
Under ground	*C*	40.3	4100	47
	Springs	6.5	500	37
	Cantus	4	300	35
	Sum	50.8	4900	45.2
Sum total		86	7400	40

Notes: A- Reservoirs and diversion dams and pumps; B- Traditional utility; C- Deep and semi deep wells.
Source: Agricultural Jihad Ministry, special committee in soil and water planning.

Table 6. Estimation of water productivity in agricultural sector.

Kind of product	Cultivated area (ha)	Volume of utilizations (10^3 m^3)	Production (t)	Water productivity (kg/m^3)
Wheat	2177901	13502986	6651567	0.492
Barely	622923	3426076	1615682	0.471
Sugar beet	171658	4463108	4649017	1.041
Paddy	514791	12354984	1990223	0.161
Sugarcane	36860	1474400	3195333	2.167
Grain maize	172721	1727210	1064186	0.616
Folder	619708	13633576	6498437	0.476
Cotton	186640	1493120	399244	0.267
Orchards	1874192	22490304	11774988	0.523
Total cultivation area of orchards and plant ages	7398079	86051514	52786653	0.613

Source: Agricultural Jihad Ministry, special committee in soil and water planning.

Table 7. Estimation of water rate requirement of cultivation lands and water orchard.

Kind of product	Cultivated area (ha)	Volume of utilizations with 40% efficiency (m^3/ha)	Total volume of utilizations (10^3 m^3)
Wheat	217790	6200	13502986
Barley	622923	5500	3426076
Paddy	514791	24000	12354984
Sugar beet	171658	26000	4463108
Fodder	619708	22000	13633576
Cotton	186640	8000	1493120
Cereals	154195	6000	625170
Sugarcane	36860	40000	1474400
Oily seeds	130147	6000	780882
Greens	417855	14000	5849970
Patches	252350	14000	3532900
Grain-Maize	172421	10000	1727210
Others	66138	6000	396828
Orchards	1874192	12000	22490304
Sum	7398079		86051514

Source: Agricultural Jihad Ministry, special committee in soil and water planning.

Table 8. Water footprint of the same countries.

Country	Average water footprint of the country, m^3/capita/yr	Part of footprint falling outside of the country, %
Iran	1624	18
Israel	1391	74
Kazakhstan	1774	1
Kyrgyzstan	1361	0
Russia	1858	16
Turkmenistan	1764	2
Uzbekistan	979	5

Notes: Global average water footprint = 1243 m^3/cap/yr.

4 CURRENT PROGRAMS AND LONG-TERM WATER RESOURCES DEVELOPMENT STRATEGY OF THE COUNTRY

4.1 *Current programs*

Providing an adequate water supply and sanitation to the rapidly growing urban population and developing agricultural and industrial activities is increasingly becoming a problem for governments throughout the world. The predominant approach to supplying these increasing water demands has been supply augmentation schemes. But in many developed and developing countries, the cost of developing new sources or expanding existing sources are growing and becoming more physically and economically infeasible. In this trend industrial activity also demands the expansion of urban water supply services. The water resources are threatened not only by this swiftly increasing demand but also through diminishing quality caused by pollution and saline intrusion and overexploitation of water catchments area. In recent year, saving water has been more efficient and considerable rather than developing new resources of water. In many countries this approach has been accepted from both an environmental and economical point of view as being the best solution for meeting growing water demands. In order to above mentioned reasons the government decided to change the supply –oriented approach to demand-management there are several experiments carried out by government agencies, NGO, research institute and private sectors, which is to appreciate and merit careful study and evolution into national policies, strategies and action plans:

- intermediate of some determine universities, research institutes and communities to involve entirely all purposes which have been discussed. The ministry of energy is one of the pioneer in this regard with great and simply equipped research center in Iran;
- research progressing programs carried out by researchers at the universities and technology sectors indicate that are promising to solve some significant problems in the water demand and management considering promotion of public awareness in many different aspects;
- arise many extension groups sending to the villages to promote awareness and declaring the multidisciplinary aspects of water management, which is initiated by formerly ministry of agriculture, throughout the country. In order to merging of two previous ministries namely ministry of agriculture and Jihad-e-Sazandege in the past two years the constitution of new ministry called jihad-e-agriculture initiated multi purpose program considering improvement of water demand management in 2001 up to now and still proceeds it trend. These measures require efficient participation and mobilization of the agrarian society.

Regarding to huge investment for supplying water and performance of drainage and irrigation networks it has been forecasted all necessary provisions particularly to transfer and handing over the operation and maintenance management to the real operation association and arising to economize and prevention of wasting.

The proposed regulation of optimization consumptions presented by former ministry of agriculture, energy, jihad, planning and programming management organization is approved by the council of ministers in 1995, which deal with broad aspects such as water requirement of plant, crop pattern, irrigation efficiency, and substantial revision in prevailing cultivation in more than 600 plains. On this basis should all allocation and delivery appliance in determined points be provided, as volume delivery to the farmers? In order to the lack of national funds and credits in agriculture ministry to perform the studies dealing with establishing of farmer associations and volume delivery of water to the farmers, the ministry of Energy posses sufficient means to perform above mentioned issues. All these effort would have efficient and positive results, when the social and economical and cultural problems met an appropriate solution to promote the agrarian society.

4.2 *Strategies of long-term water resources development of the country*

- the water national management should be reinforced by a conjointly management;
- based on supply and demand, comprehensive in the water recycle and the sustainable development principle and land provision in the water sheds of the country, in order to management

integrity of water recourses the mutual coordination between different economical, social, infrastructure and servicing sectors with water sector is very essential;
– the country fourth development program policy; establishment of proper mechanism to promote the factors production of productivity (Energy, Capital, Workforce, Water and Soil).

5 CONCLUSION

Through out the world poverty plays an important role in making people participates intensively in water issues, one of the most significant factors is to change of experiences on water demand management in different countries, which could account as the major pace to make efficient impact on this topic.

Regarding to long term development strategies of water resources, water supreme council with cooperation of public consumer foundations relating to coincides supply and demand management in water supply put into effect, which finally required founds should be provide through general (financial) resources to perform divers hydraulic structures whereas ministry of energy is in charge for water supreme council policies execution.

The Jihad Agriculture Ministry has already initiated the strategic planning of water sector and updating of the national water optimization consumption intensively since emerging procedure of two former ministries, also Agriculture and Jihad Sazandege Ministries. The gestation period is going on considering all peripheries of water laws, regulations, and its legislation by superior council and committees, which are responsible and directly engaged with this highly valuable and precious commodity.

Due to all undertaken policies and efforts, we approached an increasing of 10% growth in agricultural sector and hope to maintain and improve this trend by water supply and consumption optimization.

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Estimation of risks and damages from possible catastrophes at ground dams in Armenia

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ABSTRACT: Earthquakes and floods may cause dam failure, though the dams can be destroyed by the causes, which are not related to natural catastrophes. For the decision of the listed problems, it is necessary to develop and improve technologically sold methods due to the estimation of Influence of all the variety of factors upon the safety of the ground dams of Armenia.

1 INTRODUCTION

As it is known quick growth of the number of natural and technogen catastrophes has been noticed in recent years and as a result a population and economic losses in many countries and society, in general, has become more vulnerable to natural disasters. thus the dam safety issues become more actual.

The incidents at dams may happen due to: seepage in foundation, body and abutment area, internal erosion of piping, overtopping, dam slumping, slides, earthquake impact, construction drawbacks, problems at downstream toe, non-maintenance of demands of exploitation norms.

Millions of people in different countries now live in the most seismically active locations and in the areas where floods are recurred year after year. Earthquake is the main natural disasters which may bring risks of dam failures and as a result followed by flood.

The subject of this article is the research of the risks of natural and technogen catastrophes threatened to the dam safety of Armenia and formation of suggestions directed to prevention of their possible consequences and damages.

2 SEISMIC HAZARDS AND ANALYSIS OF SEISMIC STABILITY OF DAMS IN ARMENIA

Armenia is completely located in the seismically active region, and as a developing country and for that it is particularly vulnerable because of the lack of resources to withstand or recover its losses from earthquake disaster (which possibility (in terms of victims and economic losses) accounted for more than 90% of all types of disasters).

After the Spitak 1988 earthquake (M = 7.0 by Richter) the seismic hazard level of the region has been reevaluated, a revision of buildings and structures affected by Spitak earthquake (M = 7.0 by Richter) has been completed and building and structures damaged by the earthquake have been restored and reinforced up to required level of seismic resistance according to Seismic Codes developed in Armenia. A complete reevaluation of the seismic resistance of the whole area of Armenia has been made.

The seismicity of Armenia according to the new map of seismic hazard is considerably higher than that defined many years ago when the dams were built. At present in Armenia there are 82 dams, most of which are embankment (ground) ones.

Most of dams of Armenian were built decades years ago, according to the Seismic Code operating in that time, therefore it is essential to re- estimation their seismic resistance (Tab. 1)

3 DENSITY OF POPULATION OF ARMENIA AND DAMS

Armenia is one of the countries with high density of population (108 men/km^2) and with location of dams in high land area at heights 1500–2500 m above the sea level (Fig. 1).

The majority of these dams were built in the 1960s and the 1970s.

Table 1. Data on Armenian dams (the data on seismic stability of dams with the height of 15 m is presented).

Dam	Type rank	Foundation rank	Design seismic resistance	Seismicity according to Armenian	Seismic code
Gekhi	EE	2	1	8	8–9
Spandarian	EE	1	1	8	8–9
Azat	EE	1	11	8	9 & above
Her-her	ER	3	1	8	8–9
Tolors	EE	1	1	8	9 & above
Jogaz	EE	1	111	7	up to 8
Akhurian	G	3	1	8	8–9
Aparan	EE	2	1	8	up to 8
Kechout	ER	3	1	8	8–9
Hakhoum	EE	1	111	7	up to 8
Tavoush	EE	3	1	8	8–9
Sevaberd	ER	2	11	8	9 & above
Sovietashen	ER	3	11	8	up to 8
David-Beck	ER	3	1	8	9 & above
Shamb	ER	2	11	8	8–9
Tavshiot	EE	3	11	7	up to 8
Aygedzor	ER	3	1	7	up to 8
Kakavadzor	ER	3	1	8	up to 8
Angegakot	ER	3	11	7	8–9
Karnut	EE	3	1	8	9 & above
Khalavar	EE	3	1	8	up to 8
Mantash	EE	3	1	8	9 & above
Sarnakhpiur	ER	2	111	8	9 & above
Erevanian	EE	3	11	8	8–9
Arkel	ER	3	–	8	8–9
Karaglukh	EE	4	1	8	8–9
Gndevaz	EE	3	–	8	up to 8
Shenik	EE	4	1	8	8–9
Agavnadzour	EE	3	1	8	8–9
Sasnashen	EE	4	11	8	8–9
Gegarkounik	EE	3	111	8	8–9
Vardakar	EE	4	1	8	9 & above
Arpilich	G	4	1	8	8–9

EE—embankment earthfill; ER—embankment rockfill; G—Gravity.

3.1 *Statistics of incidents and case of dam failure*

During recent three decades due to settlement, seepage and erosion problems four complete dam failures had occurred in Armenia: Agarak (1978), Dzorakbyur (1979), Marmarik (1974) and Artik (1994). The failure of Agarak dam caused an inundation of a village situated not far from the dam.

The failure of Artik dam caused the death of two people.

Armenian dams may be divided into two categories:

– 34 dams higher than 15 m,
– 48 rural dams lower than 15 m.

Most of rural dams pose a high hazard to the downstream population, because they had been built by local community without technical assistance of any qualified specialists.

Most small dams in Armenia were built during 50–70 years of last century when there was no dam safety control and measuring equipment.

On Tab. 2 information on incidents at dams is presented.

The main risks which cause different incidents at dams are:

– Dam body instability;
– Spillway inadequacy to discharge possible highest flood;
– Seepage and erosion of dam body and foundation;
– Dam crests settlement.

On Fig. 2 the statistics of incidents on dams of Armenia is presented.

3.2 *The exploitation characteristics of dams during spitak eatrhquake (1984, m-7 richter scale)*

The result of dams' inspection after earthquakes gave materials of great interest. The dams were inspected and different damages were revealed on 20 of them. The uncompleted dam Getik was designed for earthquakes with the intensity of 8. According to the scheme of isoseists of the Spitak earthquake the dam site had experienced the

Table 2. Dam failures in Armenia in 1974, 1976, 1978 and 1994.

Dam name	Dam type	Dam failure year	Age of dam (yar)	Cause at failure and consequences
Agarak	Earth fill	1978	20	Leakage, sloughing, crest settlement and overtop ping. One village flooded
Marmarik	Earth fill	1974	End of construction reservoir empty	Excessive pore pressure build-up, leakage, sloughing
Artik	Earth fill	1994	15	Progressive failure of upstream slope and consequent overtopping Two villages flooded, 2 casualties, many losses
Dzorakhpyur	Earth fill	1976	6	Leakage, upstream sloughing, overtopping

impact with the intensity of 9. The right bank site of the dam is situated nearly the landslide. During the earthquake there was a large horizontal dislocation of the fragment of the landslide with the volume of 10×10^6 m^3.

Karnout dam with a capacity of 25×10^6 m^3 is situated at the site of the seismicity 8 according to Seismic Code—81. The tower of floodgate was displaced on 15 cm after the Spitak earthquake and numerous ruptures were formed on the concrete facing in the right—bank site of the dam.

Khalavar dam built in 1982 and designed for intensity 8 is situated in the same zone according to isoseists of the Spitak earthquake. On the lower slope of the dam cracks were formed with opening up to 5 cm and visible depth up to 0,6 m.

Aparan dam is situated at the site with the seismicity 8 and according to the Spitak earthquake isoseists the intensity was 7. During the visual observation the cracks with the width up to 3 mm were found on the walls of the floodgate.

Jogaz dam is designed with intensitpy 8 according Seismic Code—81, and it coincides with the seismicity of that site which is also 8. According the Spitak earthquake isoseist the intensity of Spitak earthquake in that area was 6. At the entrance part of the irrigation floodgate there was a large dislocation of the walls side in the results of earthquake influence.

3.3 *Dams seismic vulnerability evaluation*

The evaluation of the dams' vulnerability is one of the most important problems of the seismic risk reduction. The consequences the latest earthquakes in India, China and Japan showed one more time that most of the dams are not resistant enough to withstand seismic forces at expected maximum level.

In Fig. 3 and Fig. 4 map of seismic risks and dams of the central region of Armenia are presented.

Among 33 dams of Armenia with the height more than 15 m, 9 ones are situated in the seismic zone with g = 0.4, 21 ones—in the seismic zone with g = 0.3, 3 ones—in the seismic zone with g = 0.2.

3.4 *Dams seismic vulnerability evaluation*

Among 33 dams of Armenia with the height more than 15 m, vulnerability for 20 ones was calculated, connected with the dams' stability during the seismic influence. Among 23 calculated dams:

- 6 ones have high vulnerability (Azat, Tavshout, Sovetashen, Jogaz, Karnout, Sarnakhpyur);
- 9 ones have medium vulnerability (Vardakar, Aparan, Mantash, Hakhoum, Gekhardalich, Kakavadzor-2, Her-her, Kechout):
- 6 ones have low vulnerability (Akhurian, Arpilich, Aygedzor, Khalavar, Tavoush, David-beck).

3.5 *Dam failure and risk evaluation*

The risk of dam failure cannot be reduced to zero.

Generally the failure of dam is a high hazard, but if there is no hazard to life or -property within the inundation zone of the dam it may present a small risk.

The risk factors that can cause dam failure are reevaluated as high risks when people or property is threatened.

These risk factors can be classified and grouped into one of four categories:

- natural factors;
- structure factors;
- human factors;
- operating factors.

Natural risks such as earthquakes, natural floods, floods caused by dam failure, landslides are most crucial factors of risk.

The consequences of a flood due to earthquake, which causes dam failure usually considerably exceeds the consequences of expected natural

flood at maximum level, therefore, residencies and businesses, that would escape natural flooding, can be at more extreme risk from dam failure flooding.

Structural factors are the condition of the dam and its foundation, the abutment conditions, seepage potential dam material characteristics, characteristics and conditions of spillway structures, water outlets, inlets and gates conditions, cracking, settlement and slides.

All sorts of other human and operation factors should be included in risk analyses.

Vandalism cannot be excluded and is a problem faced by many dam owners.

The existing large range of natural, structural and human factors (the probability of terrorism can't be excluded) increase the probability of dam failure and losses of the people and property.

There are many different qualitative as well as quantitative approaches in defining classes of dam risk.

It is evident, that for qualitative risk assessment it is necessary to implement an extensive monitoring, collect a large volume of information and perform a statistical analysis.

Having studied extensive international experience in this filed the experts have given a weighting to each of dams coming out from the sizes of consequences of risks and possibility of their repetitiveness.

Of course, in assigning weights there is a subjective element, though the acquired wide experience makes it somehow possible.

Generally for different structure of dams the most characteristic problems were identified:

- Crest problem: ruts, along crest, excessive settlement, sinkhole, longitudinal cracks, and transverse cracking.

  The largest weight was given to "transverse cracking"
- Upstream slope problem: local settlement, inadequate erosion problem, too steep a slop, a rapid drawdawn of the reservoir.

  The largest weight was given to "rapid drawdown"
- Downstream slope problem: seepage through the dam, settlement of embankment, loss of embankment material strength from excessive seepage, internal erosion, seepage water existing from a water boil, standing water at the downstream toe.

  The largest weight was given to "water boil"
- Outlet problem: inoperable low-level outlet, deterioration of concrete, conduit is poor condition, corrosion, inoperable gate.

  The largest weight was given to "inoperable gate"
- Spillway problem: eroded channel, blocked channel, inadequate erosion protection, too steep a gradient.

  The largest weight was given to "eroded channel".

The corresponding weights have been given also to seismicity of the territory, hydrology, downstream population, property etc.

The largest weight was given to the threat to population in case of inundation risk.

To predict beforehand exactly how much area would be flooded presents difficulties.

The Software "Volna-2" and GIS technologies have been used for defining how much area may be flooded in Armenia in the results of dam failure.

As a result of works the risk for 42 dams of Central Region of Armenia has been evaluated.

As a result it was revealed that 42% of dams have high risk of destruction, 35%—have moderate risk and 23% have low risk.

3.6 *Summary and conclusion*

The main causes of incidents on dams in the world are based on the lack of information by the experts on regular monitoring of the technical status of the dams.

Sudden total failure of a dam as a result of earthquake, destruction of dams followed by inundation of population in downstream area, forces to pay special attention to the seismicity of the region the dams are built in.

Many of the large and small dams in Armenia are located in zones of relatively high seismicity.

The seismicity in the territory of Armenia, according to the new map of seismic hazard, is considerably higher than that defined many years ado, when the dams were constructed.

Overwhelming majority of rural population living in the dam-influenced area has a poor awareness on both positive and negative aspects stipulated by the presence of reservoirs.

Coming out of the fact that the earthquakes cannot be predicted in the near future, the most correct will be the giving of the assessment of the dam risk, which will allow then to install water alarm system for more dangerous dams.

In this case a large number of people could be saved from flood caused by possible destruction of a dam and economic losses will be decreased.

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South East Europe hydropower project, targeted to improve water resource management for a growing renewable energy production

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ABSTRACT: The *SEE HYDROPOWER* project, financed by the South-East Transnational Cooperation Programme (European Union), aims to a sustainable exploitation of the water resources concerning hydropower production in South-East Europe countries, looking up to renewable energy sources development, preserving environmental quality and preventing flood risk.

Hydropower is the most important renewable resource for energy production in the South-East Europe countries but creates local ecological impacts.

The project gives a strong contribution to the integration between the *Water Framework Directive (WFD) and the Renewable Energy Sources Directive (RES-e)* in the involved countries.

Objective of the SEE HYDROPOWER deals with the promotion of hydro-energy production in South-East Europe countries, by the optimization of water resources exploitation, in a compatible way with other water users, using friendly approaches regarding the environment.

Competition between water users (for drinking, irrigation, industrial processes, power generation, etc.) is becoming a serious problem and there is a strong need of a more accurate planning and management optimization of the natural resources.

The project defines specific needs and test methodologies & tools, in order to help public authorities to take decisions about planning and management of water and hydropower concessions, taking into account the environmental sustainability of natural resources. In the same time the project investigates strategies and methods for preserving river ecosystem considering the required environmental flow, riparian habitat quality, migratory fishes and other related environmental issues.

The paper presents the preliminary results achieved in the pilot river basins Ialomita and Prut and the project work packages results concerning Romanian legislation in the field of hydropower water use, the environmental flow assessment and the level of Water Framework Directive implementation in our country.

1 SOUTH EAST EUROPE PROGRAMME DESCRIPTION

The South East Europe Programme is a unique instrument which, in the framework of the Regional Policy's Territorial Cooperation Objective, aims to improve integration and competitiveness in an area which is as complex as it is diverse.

The program is supporting projects developed within four Priority Axes: Innovation, Environment, Accessibility, and Sustainable Growth Areas—in line with the Lisbon and Gothenburg priorities, and is also contributing to the integration process of the non-EU member states.

The South East Europe Programme aims to develop transnational partnerships on matters of strategic importance, in order to improve the territorial, economic and social integration process and to contribute to cohesion, stability and competitiveness of the region. For this purpose, the Programme seeks to realize high quality, result oriented projects of strategic character, relevant for the program area.

The South East Europe Programme helps to promote better integration between the Member States, candidate and potential candidate countries and neighbouring countries. Regional cooperation in South East Europe is essential, regardless of the different stage of integration of the various countries. The stability, prosperity and security of the region are of significant interest to the EU.

Financial instruments of the program are: ERDF, IPA and ENPI.

http://www.southeast-europe.net/en/ is the official site of the South East Europe Programme.

The project SEE HYDROPOWER is developed within Priority Axes 2: Protection and Improvement of the Environment and the Area of Intervention is: Eol 2.4 Promote energy and resources efficiency.

2 SEE HYDROPOWER PROJECT DESCRIPTION

"SEE HYDROPOWER project—targeted to improve water resource management for a growing renewable energy production"—started on June 2009 covers a period of tree years and is financed by the South-East Transnational Cooperation Programme (EU).

www.seehydropower.eu is the official web site of the project.

2.1 *Project background*

SEE HYDROPOWER is based on the European Directive on the promotion of Electricity from Renewable Energy Sources (RES-e) in respect to the Kyoto protocol targets, that aims to establish an overall binding target of 20% share of renewable energy sources in energy consumption and a 10% binding minimum target for bio-fuels in transport to be achieved by each Member State, as well as binding national targets by 2020 in line with the overall EU target of 20%.

2.2 *Problems or challenges to be addressed*

The project aims to a sustainable exploitation of water concerning hydropower production in SEE countries, looking up to renewable energy sources development, preserving environmental quality and preventing flood risk.

The SEE HYDROPOWER project promotes the optimal use of the natural water resources for different uses including hydropower production, in order to face the increasing of the regional power demand.

The project brings an important contribution to the implementation of the Water Framework Directive (WFD) and the Renewable Energy Sources Directive (RES-e) in the SEE countries.

Another issue on witch the Project is addresses is the contrast between the different approaches of the Public Administration and Environmental associations on one side and the Hydropower producers on the other side, for the exploitation of water resources.

2.3 *Project partners*

The project partnership significantly represents not only the SEE countries but also includes a well balanced mixing of public administrations (5 partners) and agencies (2 partners) ruling hydropower development and related water bodies conservation; and scientific & research institutions (5 partners) having the most advanced technology and standard background applied to water management and hydropower generation. All partners have on field specific experience in their own country issues and represent a wider local/regional partnership not "formally" involved in the project but strictly connected to policy and water management. Target groups representatives from SEE countries are reinforced by the presence of several observers (6 partners).

Project Partners (PPs) are presented below:

- Public Administrations:
  BELL—province of Belluno, Italy; STYRIA—Government of Styria, Austria; APELLE—National Administration "Romanian Waters", Romania; SERRES—Prefecture of Serres Province, Greece; ARTA—Prefecture of Arta Province, Greece
- Agencies:
  ARPAV—Regional Land Safety Department, Italy; MOP—Ministry of the Environment and Spatial Planning, Slovenia
- Scientific and Research Institutions:
  ERSE—Ricerca sul Sistema Electrico Sp A, Italy; TUG—Graz University of Technology, Austria; UL—University of Ljubljana, Slovenia; POLI-B,—University "Politehnica" Bucharest, Romania; MOLD—Technical University of Moldova, Republic of Moldova.
- Observer partners:
  AHP—Austrian Hydro Power, Austria; ELEM—Macedonian Power Plants, Macedonia; ESHA—European Small Hydropower Association (EU); ASSOELE—National Association of Electric Utilities, Italy; ENERGAP—Energy Agency of Podravje, Slovenia; APER—Association of Renewable Energy Producers, Italy; UNESCO; Regione Veneto.

2.4 *Stakeholders*

The project targets all groups and persons "around" the use of "hydropower". The most important target groups are:

1. Regional/national organized and transnational decision makers, representing a lot of entities in their countries;
2. Policy decision makers in the participating countries and SEE-area;
3. Research centres;
4. Universities;
5. Professional disseminators and multipliers such as energy agencies;
6. Environmental organizations
7. Investors
8. Small and medium enterprises
9. Non profit organizations
10. Hydropower associations

Stakeholders already detected in partner countries are: Regione Lombardi, Provincia Autonoma di Trento, Regione Veneto, Enel, Edison,

AEM, Engadiner Kraftwerke, Verbund Austrian Hydropower AG, Soške Elektrarne, Dravske Elektrarne, Slovenian Energy Directorate, among others.

3 PROJECT CONTEXT

Hydropower is the most important renewable source for energy production in the SEE countries but creates ecological impacts on a local scale. If on one hand, hydropower production has to be maintained and likely increased depending on demand trend and RES-e Directive implementation; on the other hand, producing hydropower often involves severe hydrological changes, damaging the connectivity of water bodies and river ecosystems.

The project will contribute to the integration of both WFD and RES-e Directive in the targeted countries.

While the potential for the use of environmental friendly technologies and the assets for future economic and social development are the strong points of the area, inherited environmental damage has to be addressed as well in the SEE countries.

The 2001/77/EC (RES-e Directive) was transposed in the Romanian legislation and targets have been set by the National Energy Strategy for 2007–2020, on the use of RES-e at different rates from the internal gross consumption of electric energy for 2012, 2015 and 2020.

Romania adopted the system of compulsory quotes combined with green certificates trading system. The sale price of the green certificates has to fit between certain limits that are legally regulated by Electricity Law 220/2008 amended by Law 139/2010.

The National Energy Strategy for 2007–2020 takes in consideration a sustainable development of the energetic field.

The 2000/60/CE Directive (WFD) requires that all member States to reach by 2015 a "good" ecological status for water bodies. Administrators face an increasing water demand, but lack reliable tools to evaluate the effects of water withdrawal on river systems. Competition among water users (for drinking, irrigation, industrial processes, power generation, etc.) is becoming a serious problem, and it requires a more accurate planning and management for the optimization of the resources.

4 MAIN OBJECTIVES

Objectives of the SEE HYDROPOWER deal with the promotion of hydro energy production in SEE countries, by the optimization of water resource exploitation, in a compatible way with other water users, using a friendly environmental approach.

SEE HYDROPOWER defines specific needs and tests methodologies and tools, in order to help public bodies to take decisions about planning and management of water resources and hydropower concessions, in order to satisfy all multi-purposes uses taking into account the sustainability of the natural ecosystem and the flood risk.

Investigations are carried out to define common strategies and methods for preserving river with particular concerns to aquatic ecosystem, considering the Environmental Flow, macro-habitat quality, migratory fishes and related environmental issues.

Define and test common strategies and methods to promote the development of Small Hydropower Plants, distributed in the territory recovering the remaining water power availability, and using an "ecological approach".

5 AIMS OF THE PROJECT

The Project aims are:

- To investigate the stage of transposition and implementation of the WFD and RES-e Directive, of planning and management of water resources and hydropower plants, and to identify multi-criteria decision support framework and dynamic operation of reservoirs in the partner countries. This will be achieved by defining polices, methodologies and tools, for a better water and hydropower planning and management;
- To determine the stage of implementation of the Environmental Flow according to WFD, to identify methodologies related to water quality and biological continuity and strategies for preserving river ecosystems. This will be achieved by establishes commune criteria for preserving water bodies ecosystems;
- To identify common strategies to improve the exploitation of Small Hydropower Plants (SHPs) potential, application of informatics tools, dissemination of SHPs opportunities among investors. This will be achieved by assess common strategies to improve SHP implementation;
- To test methodologies and tools in pilot regions of five countries: Piave basin (Italy), Mur basin (Austria), Ialomita and Prut basins (Romania), Serres region (Greece), Drava basin (Slovenia) to assess promising methodologies to improve water resource management for growing hydropower energy production in the SEE countries. This will be achieved by tests studies in pilot case river basins of the partner countries;
- The SEE Hydropower project has been developed with special attention to motivate communication between technicians, policy

makers and public managers from the South East European countries. This will be achieved by promotion and dissemination of project outcomes among target groups all over the SEE Region countries.

The official website of the project www.seehydropower.eu is the key instrument for helping to achieve the objectives of the project.

Attending national and international scientific events is another instrument for dissemination.

6 EXPECTED PROJECT OUTPUTS

- Reliable multi-criteria methodology to support water planning and management for hydropower production;
- Tested dynamic reservoir management to improved power production and mitigate flood risk;
- Guidelines of best practices to preserve river ecosystems, with special concerns to Environmental Flow, macro-habitat and migratory fishes;
- Methods and strategies (GIS database, site public cadastre, software) to improve eco-hydropower production by means of SHP;
- Communication among SEE countries concerning hydropower production, contributing to the implementation and integration between WFD and RES-e Directives.

7 HYDROPOWER REGARDED AS RES-E ALTERNATIVE—HYDROMORPHOLOGICAL PRESSURE OR SOLUTION FOR THE ACHIEVEMENT OF THE ENVIRONMENTAL OBJECTIVES?

7.1 *WFD and other water related Directives—Intersection of environmental objectives*

The Water Framework Directive (2000/60/CEE), the Directive 79/409/EEC on the conservation of wild birds (amended by 2009/147/EEC) and the Directive 92/43/EEC on the conservation of natural habitats and of wild fauna and flora are the main Directive which define the policy in water field of the EU community. Each of them has its own environmental objectives.

The aim of the Directive 2000/60/EC "establishing a framework for community action in the field of water policy" (WFD) is "good ecological status" or, where relevant, "good ecological potential" for all water bodies in the Member States.

Article 4 of the WFD sets environmental objectives for the different categories of water bodies and for protected areas (surface and groundwater bodies).

The environmental objectives for protected areas and the deadline for their achievement are set by Article 4 (1c) which applies to all the protected areas listed in the register of protected areas Natura 2000 with the content described in Article 6 "Register of protected areas" of the Directive 2000/60/EC.

The Annex IV of the Directive 2000/60/EC contains five types of protected areas which form the register of protected areas. Among these, are included: "areas designated for the protection of habitats or species where the maintenance or improvement of the status of water is an important factor in their protection, including relevant Natura 2000sites designated under Directive 92/43/EEC and Directive 79/409/EEC".

Natura 2000 is an ecological network of protected areas which aims to maintain a favourable conservation status for important habitats (listed in Annex I of the Habitats Directive) and species types (listed in Annex II of the Habitats Directive and in Annex I of the Birds Directive) in Europe.

Directive 92/43/EEC on the conservation of natural habitats and of wild fauna and flora (amended by Directive 97/62/EC) "contributes towards ensuring bio-diversity through the conservation of natural habitats and of wild fauna and flora in the European territory of the Member States", with "a coherent European ecological network of special areas of conservation which shall be set up under the title of Natura 2000".

Whereas, the main aim of the Habitats Directive being to promote the maintenance of biodiversity, taking account of economic, social, cultural and regional requirements, this Directive makes a contribution to the general objective of sustainable development.

Directive 2009/147/EC on the conservation of wild birds (amended by Directive 2009/147/EC) relates to "preserve, maintain or re-establish a sufficient diversity and area of habitats for all the species of naturally occurring birds, their eggs, nests and habitats, in the wild state in the European territory of the Member States".

The Habitats Directive (together with the Birds Directive) represents the cornerstone of Europe's nature conservation policy. It is built around two pillars: the Natura 2000 network of protected areas and the strict system of species protection. Overall the directive protects over 1.000 animals and plant species and over 200 so called "habitat types" (e.g. special types of forests, meadows, wetlands, etc.), which are of European importance.

By the Position Paper—Chapter 22 "Environment" Romania committed to implement

the environmental objectives and the rest of the requirement and provisions of the three European Directives mentioned above, and are transposed in the Romanian legislation by Water Law no. 107/1996, with subsequent amendments, and GEO no. 57/2007 regarding the regime of natural protected areas, conservation of natural habitats, wild flora and fauna.

7.2 *Implementation of water related EU Directives in Romania*

The WFD is transposed into the Romanian legislation by Water Law 107/1996 amended by: Law 310/2004, Law 112/2006 and Governmental Emergency Ordinance no. 3/2010.

In order to implement the objectives of the EU directives and particularly an integrated water management at the basin level, a very good communication with all the local and national administrative and environmental authorities had to be accomplished.

It is accepted by all parties involved in water domain that a sustainable qualitative and quantitative water management has to include, on one hand, draught and flood management and, on another hand, conservation of the aquatic ecosystem biodiversity.

In order to achieve this goal, in accordance with the Water law, River Basin Development and Management Plan (RBDMP) should become an instrument for water planning and management, including River Basin Development Plan (RBDP) and River Basin Management Plan (RBMP), elaborated and reviewed by the National Administration "Romanian Water" (Romanian Water Authority) with the consent of the River Basin Committees and the approval of the Government and are subject of public debates, all parties involved being invited.

The National Administration "Romanian Waters" develops a good tool for providing quality network database which may be used as input in water resources and water management simulation models Thus, some models are going to be operated (e.g. MIKE BASIN) using the GIS interface. The GIS system was mainly used in implementing Water Framework Directive. However, some aspects connected to other water related EU Directives were included in GIS system.

In the future, an interface for the cadastral structure using GIS data-base shall be developed, aiming to clarify important executive issues, such as: water right or concession granting and supervision.

7.3 *Implementation of RES-e Directive in Romania and environmental impact assessment*

2001/77/EC Directive (RES-e Directive) on the promotion of electricity produced from renewable energy was transposed in the Romanian legislation and its targets were regulated by the Governmental Decision no. 443/2003 and later improved by the Governmental Decision no. 958/2005.

The Electricity Law 13/2007 transposes the provisions of the Directive 2003/54/EC concerning common rules for the internal market in electricity and (repealing the Directive 96/92/EC) and the provisions of Directive 2004/8/EC on the promotion of co-generation based on a useful heat demand in the internal energy market.

Romania is currently implementing the provisions of the Directive 2009/28/EC regarding the promotion of RES-e until 2020 (repealing the Directive 2001/77/CE already transposed in national legislation).

In Romania, targets have been set on the use of RES-e produced at a rate of 33% from internal gross consumption of electric energy by 2012, and by Governmental Decision no. 1069/2007—regarding the approval of the National Energy Strategy for 2007–2020—the targets for using RES-e were set at a rate of 35% from internal gross consumption of electric energy by 2015 and 38% by 2020.

The National Energy Strategy for 2007–2020 takes in consideration a sustainable development of the energetic field: increasing energetic efficiency, promoting the energy produced of renewable sources, encouraging the activities of research—development and disseminating the applicable results of the researches.

RES energy produced in hydropower facilities with an installed power less than or equal to 10 MW commissioned or modernized between 2004 and today, will benefit of the promotion system, as stipulated by the Guidelines issued by the Romanian Energy Regulatory Authority (RERA) for the producer of electricity from renewable energy sources in 2009. Romania has adopted the mandatory quota system combined with the trade system with minimum and maximum price limits legally set up for the Green Certificates (GC).

In the national legislation it is not clearly specified that Studies for Environmental Impact will be made differently, depending on power installed in hydropower plants (less than or greater then 10 MW).

The legislation establishes competent authorities and administrative procedures in the field of hydropower water use.

The administrative procedures (licenses and permits) and the steps to be followed (guidelines) in order to implement Small Hydropower Plants (SHPs) in Romania are regulated by different specific legal acts.

In Romania, the competent authority in the energy field is the Romanian Energy Regulatory Authority (RERA)—institution under direct subordination of the Government and the Prime Minister.

The concession procedure involves two aspects:

a. The concession contract for the use of the SHP settlement can be signed for a period no longer then 49 years with an extension right no longer then a half of the period of the first contract.
b. The concession of the water with energetic potential is administrated by National Administration "Romanian Waters" (NARW) which leases the river stretch/sector. Therefore water management licenses and permits should be obtained from NARW.

Regarding the environmental issues, the Romanian legislation specifies that Studies for Environmental Impact shall be elaborated for all hydropower constructions. The operator/user must have an environmental accord/agreement issued by local environment authority, according to Governmental Order no. 860/2002 regarding the environmental impact of hydropower facilities in line with the Environmental Impact Assessment (EIA) Directive 85/337/EEC (of 1985 with amendments in 1997, in 2003 and in 2009) and the Strategic Environmental Assessment (SEA) Directive 2001/42/EC regarding energy produced in hydropower facilities. The common principle of both Directives is to ensure that plans, programmes and projects likely to have significant effects on the environment are made subject to an environmental assessment, prior to their approval or authorisation. Consultation with the public is a key feature of environmental assessment procedures.

In addition to the already mentioned issues, the concession procedure for public goods must be applied following economic/financial provisions according to the legal procedure for public acquisitions.

7.4 *Environmental flow implementation—Romanian approach*

The modern concepts of Integrated Water Management and Sustainable Water Resources Development provided by the European Directives on water field (Water Framework Directive 60/2000/EC) and the Romanian specific legislation (Water Law 107/1996) combine the aspect of rational use of the water resources with the protection of the aquatic ecosystem.

The ecological river restoration is one of the main objectives of these concepts that aim to:

– providing the proper flow along the river course to protect the aquatic ecosystems and the associated terrestrial ecosystems;
– providing the lateral and longitudinal connectivity of the flows on the watercourses to ensure both upstream-downstream and downstream-upstream fisheries migration.

A very important phase in the complex process of ecological restoration is to ensure the flow for the ecological protection of the water course, respectively minimum flow in any cross section to ensure the natural living conditions of the existing and future aquatic ecosystems.

Currently, in Romania (Appendix No. 1 of the Water Law) the flow that must be ensured downstream of dammed sections should comply with the conditions specified within the water management permits/issues should cover:

– the sustainable maintenance of the aquatic ecosystems life conditions on downstream (minimum flow/ecological flow) and
– the quantity of water required by the water uses downstream including the flow for aquatic ecosystem protection (reserved flow/environmental flow).

There is no legal regulation on computing the Environmental Flow in our country.

Before the promulgation of Water Law 107/1996, the determination of the minimum flow value was made by the designer of hydrotechnical work, without any consideration for ensuring the living conditions for the aquatic ecosistems. In Article 41, paragraph 2 of this law, it is specified that the minimum flow, mandatory in riverbeds, are established by the National Administration "Romanian Waters".

One common situation in some rivers may be the one created by the existence of hydrotechical works, involving the determination of a minimum flow in the natural water course, where the upstream flow is already polluted. In this case providing the minimum flow may raise a question: is this determined flow the adequate one for the existing situation or the flow before the start of the anthropogenic pollution process of the water course? Naturally, the answer to this question is one that takes into account the progressive restoration of the environmental quality (considering the acquatic system) first of all by reducing the pollution from the sources.

Therefore, the minimum flow needed in the river bed downstream of hydro-technical works, has to ensure:

– protection of aquatic ecosystems and adjacent ecosystems of the watercourse;
– groundwater supply;
– water dilution, respectively its quality protection.

Some authors (Galie, 2006) have calculated the flows necessary in the river bed based on relationships where the basic element was the dilution flow, naturally considering that between the two values there is a direct link. The resulting equations, applicable to river basins with areas less than 3000 km^2, are:

$Q_a = Q_{95\%} + 0.1$ for $Q_{95\%} > 200$ l/s

$Q_a = 1.25\ Q_{95\%} + 0.05$ for $Q_{95\%} < 200$ l/s

Accepting as a rate of the dilution flow to be considered as necessary for aquatic ecosystem protection is most appropriate for the various types of hydrological regime in Romania.

A more exclusive approach, especially suitable for large river basins developed on all the landforms, may take into account flows resulting from the probability curve of daily average values. In this case the flows with 95% probability will be less than the dilution flow but more adequate for minimum flow regime for such rivers. In this respect, it should be noted that dam design in Romania used to provide a minimum flow rate (ecological flow) that had very low values, comparable to the $Q_{95\%}$ values from the probability curve of average daily minimum flows.

Thus, within the National Institute of Hydrology and Water Management the Flow Duration Curves (FDC) is used for computing the minimum flow.

The duration curves take into account series of values for a period of time (monthly or daily average) and not only the extreme (minimum) annual or seasonal values.

Flow Duration Curve (Stanciu & Neculau, 2009) is a graphical representation of the river flows depending on exceeding frequency and is determined based on long and representative data series being used in water management applications.

The determination of the environmental flow is one of the applications of the Flow Duration Curve. The ecological quality of the rivers should be maintained by sustaining a minimum flow as described above.

The minimum flow needed to be sustained in the river bed can be considered the flow with 95% probability from the Flow Duration Curve of the average daily flows (DFDC) or of the average monthly flows (MFDC).

For large rivers is indicated $Q_{95\%}$ of DFDC and for the small rivers (not dry), $Q_{95\%}$ of MFDC.

In the first version of River Basin Management Plan, based on the available studies developed by research institutes, the environmental flow was considered the minimum between $Q_{95\%}$ (yearly minimum monthly mean discharge with 95% probability) and 10% out of the average mean discharge over many years.

New approach for computing the environmental flow takes into consideration relevant international practices.

7.5 *Assessment of water bodies status in Romania*

In Romania, the National Institute of Research—Development for Environmental Protection elaborated WFD compliant classification systems for the purpose of ecological status/potential assessment. The principle "one out all out" was applied. The assessment was based on: monitoring data, water bodies grouping principle and updated risk assessment. Three classes of confidence (high, medium and low) were defined for characterization of the assessment according to the International Commission for the Protection of the Danube River (ICPDR) recommendations.

At the national level, 4 significant water management issues were identified:

- pollution with organic substances
- pollution with nutrients
- pollution with hazardous substances
- hydromorphological alterations

The main pressures on water bodies are:

- human agglomerations, especially those which do not have proper wastewater treatment plants or which do not have proper sewage collection system;
- industry, especially some installations under the provisions of the Integrated Pollution Prevention and Control Directive (IPPC), which obtained a transition period for implementation;
- agriculture, even less intensive than the average in other European countries;
- hydromorphological pressures, that generate interruptions of the longitudinal and lateral connectivity, flow abstraction/discharges.

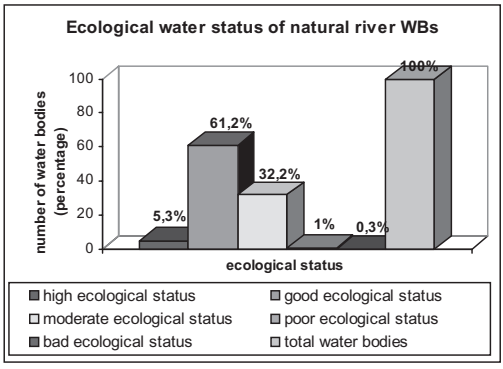
For the biological elements, analysis indicates that most of the WBs, do not reach good status due to benthic invertebrates. Organic substances, nutrients and heavy metals are the physico-chemical elements which determine failing good status.

The ecological status was defined on the basis of biological, physico-chemical and hydromorphological elements. For the river WBs, the biological elements (phytoplankton, benthic invertebrates and fish fauna), physico-chemical elements (oxygenation conditions, nutrients, acidification status and specific pollutants) were considered and for each type of WB specific values were defined.

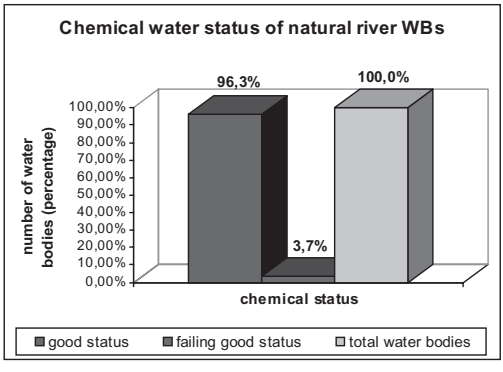
For HMWBs, the ecological potential was classified on the basis of relevant biological (benthic invertebrates for rivers and phytoplankton for reservoirs and also for both the presence and absence of fish fauna) and physico-chemical elements.

At national level, the results of the WBs assesment are described below.

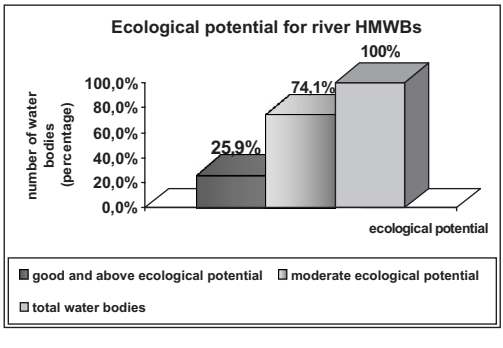
From the total of the natural river water bodies 5.3% of the rivers achieve high ecological status, 61.2% achieve good ecological status, 32.2% achieve moderate ecological status, 1% achieve poor ecological status and 0.3% of the rivers achieve bad ecological status.



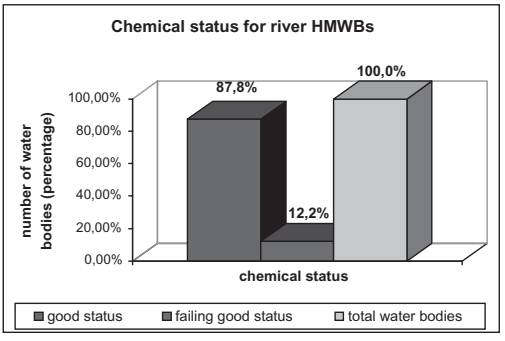
From the total of natural river water bodies (96.3%) rivers achieve good chemical status, and (3.7%) rivers do not achieve good chemical status.



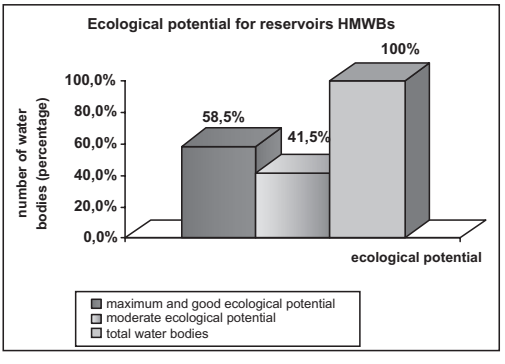
From the total of the heavily modified water bodies 25.9% of the rivers achieve good and above ecological potential, 74.1% achieve moderate ecological potential.



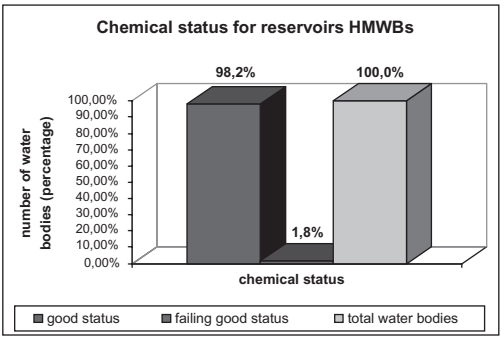
From the total of heavily modified water bodies (rivers), 87.8% rivers achieve good chemical status, and 12.2% rivers do not achieve good chemical status. For 60.4% of the rivers, the chemical status assessment was done at a medium level of confidence, and for the rest the level of confidence was low.



From the total of the reservoirs water bodies 58.5% achieve maximum and good ecological potential and 41.52% achieve moderate ecological potential.



From the total of heavily modified water bodies (reservoirs), 98.17% achieve good chemical status and 1.8% do not achieve good chemical status. For 51.2% of the reservoirs, the chemical status assessment was done at a medium level of confidence, the rest of the water bodies were assessed at a low confidence level.



Considering the results of the WBs assessment in Romania (graphical representation above) can be noticed a good chemical status for most of WBs.

Therefore, the stress comes from other pressures out of which the hydromorphological alterations (especially the sealing hydraulic works across and along the river) significantly affect the ecological status of water bodies.

In the pilot river basins Prut and Ialomita the preliminary assessment of water bodies status show all most the same rates as at the national level with insignificant deviations. A more accurate evaluation will be done in the following periods when the data collection will be completed.

7.6 *Measures concerning hydromorphological alterations—solutions to enhance the water bodies status*

The Romanian network of rivers is located in the Danube River Basin District (DRBD). At the level of DRBD there has been established management objectives described in the DRBD Management Plan.

At the national level, the environmental objectives for WBs were established. The objectives are: good ecological status and good chemical status for natural WBs and good ecological potential and good chemical status for HMWBs. Test designation of heavily modified and artificial water bodies to be reviewed every six years, meaning achieving type-specific objectives for all relevant biological (macroinvertebrates, fish fauna, phytobenthos, phytoplankton and macrophytes), physico-chemical and hydromorphological elements.

Action Program is an important chapter of the River Basin Management Plan, which includes all measures to be taken during 2010–2027, so that environmental objectives shall be achieved.

For the hydromorphological alterations the main management objectives to be implemented until 2015 for EU Member States are:

- Construction of fish migration passes and taking other measures to achieve/improve river continuity;
- Restoration, conservation and improvement of habitats and their continuity for sturgeon and other specified migratory species;
- Elaboration of a feasibility study regarding the possibility of migration upstream and downstream through the Iron Gate I & II dams for sturgeon and other important species;
- Protection, conservation and restoration of wetlands/floodplains;
- Determination of the implementation steps for restoration and reconnection of lost floodplains and wetlands;
- Implementation of the "no net-loss principle". Measures for improving longitudinal connectivity (e.g. fish migration, ecological flow) and for improving lateral connectivity (meandering, wetlands, and river bank structure diversification) were proposed.

For this purpose, in Romania were established a number of:

- 107 measures for longitudinal connectivity restoration meaning: building passageways for ichthyofauna migration, removing barriers for crossing thresholds, endowment of the bottom threshold with sloping channel for migration of the ichthyofauna, establishing the hydrological conditions for the reservoirs for providing the environmental flow;
- 79 measures for lateral connectivity restoration. Such measures are related to restoring meanders or side channels, restoring or creating new wetlands for some premises; banks and riverbed structure and habitats diversification;
- 66 measures to mitigate the effects of hydromorphological alterations which consist in restoring the riparian area structure—consolidation of vegetal area by seeding with perennial herbs, protecting banks through the plantation of trees, feasibility studies on restoring vertical connectivity in the case of draining and damming channels which were built in the area to restore land for agriculture, fish restocking with cyprinids family, banks stabilizing and unsilting and sanitation works.

Related to the measures concerning hydromorphological alterations, to propose an analysis process has been a challenge because of the difficulty in assessing technical feasibility and their impacts on the biological elements.

7.7 *Reservoirs operation in Romania*

In Romania, the exploitation and operation of reservoirs are based on the concept of integrated management at the river basin level regulated by Water Law no. 107/1996 with further modifications.

The Methodology of development and approval of the competences for the operating rules, the exploitation programs of the reservoirs, the Methodological Norms for the elaboration of the river basin operating code regulation and the Frame Rule for the reservoirs operation, dams operation and water supply intakes are approved by Order no. 76/2006 of the Ministry of Environment and Water.

In addition there are specific Ministerial Orders regarding the security of dams, small sized dams with reservoirs of local interest, fishing and recreation and 14 standard Technical Rules for Hydrotechnical Works.

According to Water Law, the holders of dams and reservoirs have the obligation to prepare and

comply with the provisions of the operating rules and to obtain water management licenses and permits.

The exploitation/operation rules are compulsory for hydrotechnical works like: dams with permanent or non-permanent reservoirs, polders, water supply intakes with or without dams, hydraulic structures for adjust and control of discharges.

Reservoirs Operating Rules contain all the necessary elements for an optimal operation:

- data supporting system and information—decision flux for water management;
- specific operating rules for different flow regimes: medium flow, low flow (dry periods) and high flow (flood events);
- responsibilities and decision-making competences of the dispatch-graphs.

In low flow periods as well as in high flow periods, the operation of reservoirs and water intakes is adapted to the requirements of the respective period, in accordance with the provisions of Drought Plans or Flood Action Plans.

Parameters that are relevant for the reservoirs operating rules are the level in the reservoir and the flow entering the reservoir.

Monitoring those parameters, specific operating rules for different flow regimes are applied.

In case of continuous diminishment of the current storage water volume (dry period) three specific situations are considered:

- Normal Situation, considered when the flow released downstream is greater than or equal to the flow of attention;
- Situation of attention/warning, appropriate to the situation when the flow released downstream is less than the flow of attention, but greater than or equal to the required minimum flow;
- Situation of restrictions, appropriate to the situation when the flow released downstream is less than the required minimum flow.

In case of dry period situation the main restriction rules to be applied are:

- it is prohibited to open outlets or bottom drain to avoid water losses from the reservoir;
- the water for the uses is released according to the indications of the dispatch-graphs/schedule in stages. and the Restriction Plan of water use in dry periods:

Stage I—gradual reduction of water used for irrigation systems;

Stage II—gradual reduction of flow allocated for fisheries, industry, farming;

Stage III—gradual reduction of flow for the population as a last resort to apply restrictions.

In case of flood forecast, actions to be taken are:

– water is released through outlets in order to decrease the flood retention volume in the reservoir;
– water for the uses is released from the reservoir according to dispatch-graphs.

The reservoir operation rules, in case of floods foresee operative measures depending on the water level in the reservoir (attention, danger and flood threshold)—previous experience or relaying on forecasted flow/discharges.

The decision-making data system for flood prevention contains information regarding the hydro-meteorological data, the capacities of the hydraulic structures, the exploitation regimes for the reservoirs, the surveillance system, etc.

The river basin plans for prevention and control of floods, hazardous meteorological phenomena and hydraulic structures accidents must be in accordance with the provisions of the reservoirs operating rules of the hydraulic structures existing in the river basin and the operating programs in case of floods.

In Romania, the Reservoirs Operating Rules are reviewed every six month by the specialists representing national water management and energy authorities in order to establish the water use schedule for each reservoir (dispatch graph).

7.8 *Promotion of Hydropower production, especially in SHPs—small environmental impact*

Accepting that hydropower production generate hydromorphological alterations, the modern practices of exploitation and operation of reservoirs and the application of the programs of measures plan to overcome this inconvenient in a such manner that the environmental objectives to be fulfilled.

In this sense, an example of good commune practice for reservoir operation in Romania is Stanca—Costesti Reservoir situated on the Prut River (trans-boundary river between Romania and Moldova), with a hydropower plant of 15 MW, being operated considering a significant ecological discharge (5 m^3/s), among other uses (water supply for population 10 m^3/s and fisheries 2.9 m^3/s).

From a total volume of 1400 mil m^3, the reservoir has a flood retention volume of 665 mil m^3 and a volume allocated for irrigation of 800 mil m^3.

The other water uses (including ensuring the ecological flow) have precedence over hydropower production. There is no special volume allocated to hydropower production. The hydropower is produced only when the volumes in the reservoir

are exceeding the volumes necessary for the other water uses.

There are specific operating rules for different flow regimes.

During low flow or dry periods the parameters defining the operating rules are:

- the level in the reservoir of 86.00 mABS (meters above Baltic Sea);
- the flow entering the reservoir is less then 40 m^3/s.

There are two distinct flows to be considered in the reservoir operation procedure: the flow necessary to meet the needs of all the users Q_n and the attention flow set up to1,2 to 1,4 from $Q_{n.}$

In this way, the river continuity has priority towards hydropower.

Once the informatics tools for hydropower potential assessment will be on place by using the latest recognised methodologies and tools for river basin determinations, the information will be more accessible for the stakeholders and potential developers of Small Hydro Plants (SHPs).

Since small hydropower projects usually have minimal reservoirs and civil construction works, they are seen as having a relatively low environmental impact compare to large hydropower plants.

The decreased environmental impact depends strongly on the balance between stream flow and power production. This can only be achieved using advanced informatics tools for water management purposes.

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Concerning stability and structural behaviour

Numerical modeling and calculus

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Prediction of concrete dam's reserve strength based on it's past loading history

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ABSTRACT: The presently reported FE based model for the analysis of old concrete gravity dams takes the structure's past loading history into account, with the structural influence of the following primary design factors included in the predictive model: (a) fatigue of the dam materials under slow cyclic loading corresponding to its performance history; (b) elastic-plastic constitutive relationship for the dam concrete, the foundation rock and the interfaces/joints in accordance with the dam loading history; and (c) crack formation and propagation in the concrete and the interfaces/joints, taking the stress degrading effects of cracked concrete into account. It has been found that: (1) the rate and extent of degradation of the concrete mechanical properties increases with the increasing numbers of drawdown/filling cycles of the reservoir; and (2) the degradation of the material properties of concrete due to the combined actions of loading/unloading cycles and ageing of concrete can still be practically significant.

1 INTRODUCTION

In what follows, a technique for complex static retrospective analysis of old concrete dams is presented, in which an attempt is made to match (as closely as possible) the material and structural model of a dam with the actual operation history of the structure. The proposed technique consists of: a) hypo-elastic (nonlinear-elastic fracture) constitutive model for concrete in the plane strain condition; b) constitutive model for interfaces; c) analysis of creep strain; d) analysis of crack occurrence and propagation within the body of a dam using discrete-cracking model and failure criterion for dam concrete; and e) cyclic- and ageing-induced degradation of stiffness and strength for dam concrete and interfaces.

2 CONSTITUTIVE MODEL FOR CONCRETE IN THE PLANE STRAIN CONDITION

2.1 *Guiding principles for the selection of the constitutive model for dam concrete*

The constitutive model presented in this section is based on the hypo-elastic (nonlinear-elastic fracture) formulation, which realistically simulates a path-dependent irreversible stress-strain relationship. The main advantage of this approach is that it can be easily implemented in any numerical analysis with the required input data for the model easily obtainable from the traditional uniaxial tests on concrete specimens. The model can account for the stress-strain relationship of concrete up to the peak stresses, including an increase in plastic volume under compression (dilatancy) which is a common feature of plain concrete as it approaches failure.

2.2 *Modification of the constitutive law for concrete under plane strain conditions*

The analysis of a gravity type dam, which constitutes a plane strain problem, is to be addressed in what follows. Noting the lack of experimental data, in the available literature, concerning the plane strain condition, one may reasonably employ a failure criterion for concrete subjected to a three-dimensional state of stresses and modify it to fit the more realistic plane strain (as opposed to previously used plane stress models) for the gravity dam problem (Lin 1995).

In the present analysis, the four-parameter failure criterion after (Hsieh et al. 1979) is employed, where:

$$a\frac{J_{2p}}{\sigma_c^2} - b\frac{\sqrt{J_{2p}}}{\sigma_c} - c\frac{\sigma_{1p}}{\sigma_c} - d\frac{I_{1p}}{\sigma_c} - 1 = 0 \qquad (1)$$

with:

$$I_{1p} = \sigma_{1p} + \sigma_{2p} + \sigma_{3p};$$

$$J_{2p} = \frac{1}{6}\left[\left(\sigma_{1p} - \sigma_{2p}\right)^2 + \left(\sigma_{2p} - \sigma_{3p}\right)^2 + \left(\sigma_{3p} - \sigma_{1p}\right)^2\right];$$

where $\sigma_{1p} \geq \sigma_{2p} \geq \sigma_{3p}$ = principal stresses corresponding to the peak state; σ_c = uniaxial compressive strength of concrete; and *a, b, c, d* = parameters which are to be determined experimentally.

In the present work, the concept of non-linearity index β is employed with it's extension to the three-dimensional stress state, as follows:

$$\beta = \frac{\sigma_i}{\sigma_{ip}} \left(i = 1,2,3\right) \tag{2}$$

where, σ_{ip} (i = 1,2,3) define the components of the peak stress vector.

The general secant stress-strain relation for transversely isotropic materials in the principal stress space is written as (Lekhnitski 1963):

$$\begin{Bmatrix} \varepsilon_1 \\ \varepsilon_2 \\ \varepsilon_3 \end{Bmatrix} = \begin{bmatrix} \frac{1}{E_1} & \frac{-\nu_2}{E_2} & \frac{-\nu_1}{E_1} \\ \frac{-\nu_2}{E_2} & \frac{1}{E_2} & \frac{-\nu_2}{E_2} \\ \frac{-\nu_1}{E_1} & \frac{-\nu_2}{E_2} & \frac{1}{E_1} \end{bmatrix} \begin{Bmatrix} \sigma_1 \\ \sigma_2 \\ \sigma_3 \end{Bmatrix} \tag{3}$$

In the case of the plane strain condition: $\varepsilon_3 = 0$. Then, the incremental form of Equation 3 would represent the tangent stress-strain relation for plane strain, provided that the material properties E_i and υ_i (i = 1,2) are substituted by the corresponding tangent values E_i and υ_i (i = 1,2). From Equation 2 one may determine σ_{ip} as (Lin & Raoof 1993):

$$\sigma_{ip} = \frac{\sigma_i}{\beta} \left(i = 1,2,3\right) \tag{4}$$

Substituting Equation 4 into Equation 1 through I_{1p} and J_{2p}, one gets:

$$a\frac{J_2}{\beta^2\sigma_c^2} - b\frac{\sqrt{J_2}}{\beta\sigma_c} - c\frac{\sigma_1}{\beta\sigma_c} - d\frac{I_1}{\beta\sigma_c} - 1 = 0 \tag{5}$$

from which, the following expression may easily be obtained for β:

$$\beta^2 + \frac{b\sqrt{J_2} + c\sigma_1 + dI_1}{\sigma_c} - a\frac{J_2}{\sigma_c^2} = 0 \tag{6}$$

The value of β may, then, be determined by solving the quadratic Equation 6.

The subsequent analyses of Compression-Compression and Tension-Compression modes of stresses for concrete, based on the above formulations, have indeed suggested that the assumed plane strain condition has a significantly more adverse effect (cf. the previously adopted plane stress models in the literature) on the behaviour of concrete within the region of Tension-Compression mode of stresses near the upstream face of a gravity dam, where the most dangerous state of stresses can occur with such critical state of stresses having the potential to cause major crack initiation and propagation within the body of a dam- hence, the importance of using the more realistic plane strain (cf. plane stress) formulations for better predictions of cracking when analyzing dams.

3 CONSTITUTIVE MODEL FOR INTERFACES

The problem of a dam-foundation interaction is of great practical importance, as it affects the overall behaviour of the structure. Special interface/joint elements can be employed to model the relative movement of adjacent boundaries at the dam-foundation interface. Interface/joint elements can also be used to simulate the behaviour of interfaces and joints which may generally exist in the body of a gravity dam. Generally, the behaviour of an interface depends on non-linear material properties of the surrounding media. In the analysis of a dam-foundation interaction problem, the material nonlinearity has to be considered. Therefore, the development of a constitutive model to simulate the non-linear behavior of interfaces is desirable. The constitutive relationship for interfaces, described in this section, is based on the hypo-elastic (nonlinear-elastic fracture) model. Using this approach, it is possible to simulate the shear stress-relative displacement curve of a shear test up to the peak shear stress, associated with which the effect of dilatancy is anticipated to occur within the joints.

A non-linear constitutive relationship for the interface elements can be written in incremental form as:

$$\begin{bmatrix} \partial\sigma_t \\ \partial\sigma_n \end{bmatrix} = \begin{bmatrix} k_s^t & 0 \\ 0 & k_n^t \end{bmatrix} \begin{bmatrix} \partial q_t \\ \partial q_n \end{bmatrix} \tag{7}$$

where k_s^t, k_n^t = the tangent values of the shear and normal components of the interface stiffness matrix which generally depend on the existing state of stresses.

In what follows, the developments of constitutive laws for the shear stress-strain and normal stress-strain relationships for interfaces are described.

The tangent shear modulus at any stage of the stress state (τ, σ_n) is defined as:

$$k_s^t = \frac{2k_s^s\left(\frac{k_s^s}{k_s^c} - A_1\right)}{A} \tag{8}$$

where

$$k_s^s = k_s^c\left(A_1 + \sqrt{A_1^2 - \beta_\tau^2}\right);\ k_s^c = \frac{k_s^0}{A};$$
$$A_1 = \frac{A - \beta_\tau(A-2)}{2}$$

In the above equations, the shear stress parameter β_τ is analogous to the non-linearity parameter β (see Equation 2), and is defined as:

$$\beta_\tau = \frac{\tau}{\tau_c} \tag{9}$$

where τ_c = shear strength of the interface.

Unknown parameters of the model to be determined are: the initial shear stiffness coefficient k_s^0, the shear strength of the interface τ_c, and the dimensionless parameter A, which relates the initial shear stiffness to the secant value of shear stiffness at failure. All these parameters are normal stress-dependent and can be generalized in accordance to the values of the normal stresses at the interface, as described below.

The initial shear stiffness coefficient k_s^0 depends on the magnitude of the normal stress σ_n acting at the interface. When analysing the effect of a weak shear seam in the foundation of the Karjan gravity dam (India), an interface model employing the following equation for defining the variation of initial shear stiffness value was used (Varadarajan & Sharma 1989):

$$K_s^0 = K\gamma_w\left(\frac{\sigma_n}{P_a}\right)^n \tag{10}$$

where K = dimensionless stiffness number; γ_w = unit weight of water (expressed in the same units as k_s^0); P_n = atmospheric pressure (expressed in the same units as σ_n); and n = shear stiffness exponent.

It should be noted that Equation 10 does not cater for the initial value of the shear stiffness for a joint with zero normal stress ($\sigma_n = 0$). Although Equation 10 is applicable to soil-structure interaction problems, it is not applicable to cases involving the bonded interface between concrete and rock or between two layers of concrete. Therefore, to account for the initial shear stiffness of an interface with zero normal stress

$$\left(K_s^0\right)^{\sigma_n=0}$$

a modification of Equation 10 is required. This is easily achieved by adding the value of initial shear stiffness associated with zero normal stress, to the right hand side of this equation, i.e.:

$$K_s^0 = \left(K_s^0\right)^{\sigma_n=0} + a\left(\frac{\sigma_n}{P_a}\right)^n \tag{11}$$

where a = stiffness parameter (expressed in the same units as k_s^0, $a = K\gamma_w$.

Equation 11 was employed by (Ge 1981) in the interface model for analysing joints in rock masses and interfaces between concrete and rock. This equation may be taken to be of a general form, which defines the variation of initial shear stiffness with respect to normal stress (Ge 1981). Therefore, this equation is employed in our analysis.

The direct shear test with zero normal stress can be used to determine the coefficient

$$\left(K_s^0\right)^{\sigma_n=0}.$$

The parameter a may be easily determined using a family of shear stress-relative displacement curves obtained from a series of shear tests with various levels of normal stresses at the interface. Often, an initial shear stiffness which is proportional to the normal stress by a linear law is adopted: thus $n = 1$.

The peak value of shear stress (τ_c) is also a function of the normal stress at the interface. The value of τ_c is expressed in terms of the interface properties using Morh-Coulomb criterion:

$$\tau_c = c + \sigma_n\tan\phi \tag{12}$$

where c = cohesion, ϕ = angle of internal friction (these parameters can also be determined from the direct shear tests for an interface). After the ultimate shear strength τ_c is reached, shear failure occurs and the value of k_s is set close to zero.

The stress-strain behavior of the interface is generally non-linear. However, the variations of normal stress-strain relationships between the ultimate compression and tensile failure are often approximated by a linear law- i.e. a constant normal stiffness k_n *is assumed*. Such an approach is adopted in the presently proposed model. Namely, it is assumed that the interfaces between a concrete dam and its foundation rock or between the successive lifts of

concrete can resist tension. For example, Upper Stillwater RCC gravity dam was designed for a minimum tensile strength of 1.24 MPa at the compacted lift lines. The tensile stress results in a relative separation of the points at the interface, with tensile failure occurring when the tensile strength of the interface is reached (i.e. when $|\sigma_n| \geq |\sigma_n^t|$). Beyond that stage, the interface can no longer resist any tension and it opens with the stresses dropping to zero. The initial shear and normal stiffness coefficients are, then, set close to zero. The magnitudes of the stresses, which have been artificially reduced to zero, are subsequently redistributed to other elements. In the compression mode, on the other hand, the same value of normal stiffness coefficient k_n is used until the maximum compression mode is reached.

In what follows, the results based on the proposed interface model are compared with certain available shear test data for interfaces (Ge 1981). The relevant interface parameters have been defined by Ge as:

$$\left(K_s^0\right)^{\sigma_n=0} = 230.0 \text{ kg/cm}^3, \quad a = 10.0 \text{ kg/cm}^3,$$
$$c = 3.1 \text{ kg/cm}^2, \quad \tan\phi = 1.1$$

Initially, the value of the parameter A was kept constant ($A = 1.8$) irrespective of variations in the magnitude of the associated normal stress. After inspecting the correlations between the theoretical predictions based on the proposed technique and experimental data, it was found that the differences between the two types of curves are basically caused by the differences in the assumed and actual values of the shear strength for the interface. However, it may generally be assumed, that the Equation 8 defines reasonably well the shape of the actual shear stress-relative displacement curve. In later stages of the analysis the theoretical shear stress-strain curves were re-evaluated by using a variable value for the parameter A. Certain differences between the theoretical and experimental curves (which are mainly caused by the differences between the theoretical and actual values of the shear strength) were found for the case of $\sigma_n = 3.08$ kg/cm^2 and $\sigma_n = 8.01$ kg/cm^2, however, reasonably good correlations between theory and test data were found for the case of $\sigma_n = 11.89$ kg/cm^2.

4 CALCULATION OF CREEP STRAINS

Both the in-situ data obtained from the observations of the dam concrete and long-term laboratory tests of the concrete samples clearly indicate that both the concrete creep and the concrete fatigue performance under external slow static cyclic loading have a significant impact on the mode of deformation of the dam-foundation system. In order for the analysis to take the concrete creep process into account, one should know the initial mode of deformation and material properties of the dam-foundation system. The calculations are carried out in three stages: a) *Stage* 1—the initial mode of deformation of the dam-foundation system is calculated; b) *Stage* 2—the dam-foundation system is analysed corresponding to the time when the strength of the dam concrete is anticipated to have reached its maximum value: this can happen within a period of 8–10 years, following the placing of concrete in the body of a dam, by which time the creep process may reasonably assumed to have been completed. Within this period, if the cracks occur in the body of the dam, the associated stresses should be analysed subject to both the cracking and the creep processes; c) *Stage* 3—the present time mode of deformation of the dam is analysed. The material properties of concrete are modified (Taliercio & Gobbi 1998) in accordance with the number of loading/unloading cycles n and time-period of dam operation t. If during the analysis there will be a crack propagation process in the body of the dam, the creep process may occur again due to stress redistribution (higher stresses) near the crack tip(s).

For the present mode, the Boltzmann-Volterra theory of linear hereditary creep is used to define the creep process in concrete. Based on this theory, the most general form of the equation defining the stress-strain state subject to creep for one-dimensional problems can be stated as follows (Tsitovich 1973):

$$\varepsilon(t) = \frac{\sigma(t)}{E(t)} + \int_0^t \frac{K(t)}{E(t)}\sigma(t)dt \tag{13}$$

where $\varepsilon(t)$ = total (elastic plus creep) strain at time t; $K(t)$ = creep function depending on age of loading and time t; $E(t)$ = modulus of elasticity of concrete depending on age of loading and varying with time t; and $\sigma(t)$ = stress at time t.

Assuming that $\varepsilon(t) = \varepsilon_{elastic}(t) + \varepsilon_{creep}(t)$, and that at any moment of time t:

$$\varepsilon_{elastic}(t) = \frac{\sigma(t)}{E(t)} \tag{14}$$

then, one will have:

$$\varepsilon_{creep}(t) = \int_0^t \frac{K(t)}{E(t)}\sigma(t)dt \tag{15}$$

Creep function can be calculated using the following equation:

$$K(t) = \delta_2 e^{\delta_1 t} \tag{16}$$

In the above equation, the coefficients δ_1 and δ_2 define the rate of a creep process and can be

determined experimentally by using the measured creep data obtaine over some period of time (Raoof et al. 2004). The following equation may be usefully employed to describe the variation of the modulus of elasticity of concrete for a period of about 180–540 days (about 0.5–1.5 years) (Alexandrovski & Bagri 1970):

$$E(t) = E_0 (1 - \beta e^{-\alpha t}) \quad (17)$$

where the ranges of coefficients α and β are 0.056–0.129 and 0.500–0.575, respectively (Alexandrovski & Bagri 1970).

In order to conduct a detailed discussion of a creep process, one needs to critically examine: a) the creep function $K(t)$; b) the coefficients δ_1 and δ_2 that are used to calculate the creep function $K(t)$ and depend on the time period over which the creep data has been measured; c) modulus of elasticity of concrete $E(t)$ and the influence of aging of concrete on the creep process; d) stress $\sigma(t)$ at any time t; e) creep recovery after unloading; and f) creep process during cyclic loading; etc.

Creep process has a drastic effect on the stress-strain curve for the concrete used in a dam, which itself dramatically influences the state of stresses in the body of a concrete dam. Figure 1 shows the total strain, which is the sum of the elastic strain and creep strain. The experimental curve has been obtained by loading a specimen at the age of 7 days (Alexandrovski & Bagri 1970). Initially, a constant value of the modulus of elasticity for concrete E was used in the calculations, however much better correlations were obtained when the variable (i.e. time-dependent) value of the modulus of elasticity of concrete $E(t)$ was used in the calculations.

Figure 2 shows the correlations for creep strain for a specimen loaded at an age of 28 days (Alexandrovski & Bagri 1970). Noteworthy is that the theoretical model predicts the stabilization period of the creep process to be about 180–540 days, while the tests indicate that the creep process occur for a much longer period of time

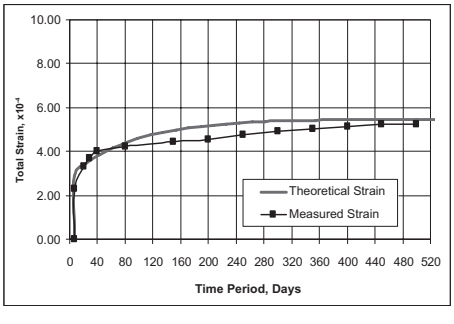

Figure 1. Correlations between the theoretical and measured strains for concrete specimen: concrete loading age of 7 days.

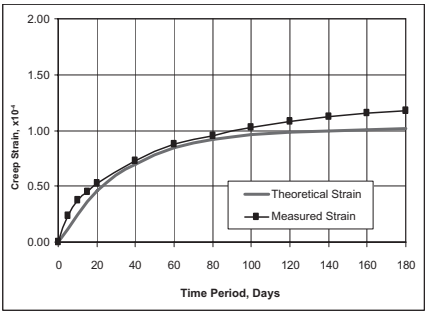

Figure 2. Correlations between the theoretical and measured strains for concrete specimen: concrete loading age of 28 days.

(it may, indeed, be resumed due to stress redistribution in the body of a dam).

5 ANALYSIS OF CRACK OCCURRENCE AND PROPAGATION

The main difficulty in modeling fracture mechanics problems arises from the presence of an infinite stress at a crack tip (Saouma & Milner 1996). This happens because a stress/strain singularity of the order of $r^{-1/2}$ (r is the distance from the crack tip) occurs in the vicinity of the crack tip. Therefore, even use of a very fine of quadratic or higher order finite and/or boundary elements is not enough to produce sufficient accuracy. In order to solve this problem, one has to employ special crack tip elements that posses the above-mentioned singularity.

The alternative method for incorporating the singularity at the crack tip is to modify isoparametric quadratic finite elements (Henshell & Shaw 1975; Barsoum 1976) and isoparametric quadratic boundary elements (Remzi 1981; Blandford et al. 1981) by shifting certain nodes on these elements. Namely, singularity elements are obtained by shifting the midpoint(s) of isoparametric quadratic elements to the quarter position nearer the crack tip, which creates the desired $r^{-1/2}$ stress/strain singularity at the crack tip. Using these elements eliminates the need to develop and use special crack tip elements, which simplifies the process of computation of the stress intensity factor at the crack tip. Furthermore, by means of using these elements, the degree of mesh refinement that is necessary to conduct at the crack tip region, in order to obtain accurate values of the stress intensity factor, is significantly reduced. In addition, by replacing the finite element region in the immediate vicinity of a crack tip by a boundary element region, the necessity to carry out the finite element mesh refinement at the crack tip region can be eliminated.

It is known that there are three modes of fracture that are based on the relative movement of

the faces of a crack: (1) the crack opening mode (mode I); (2) the crack sliding mode (mode II); and (3) the crack tearing mode (mode III). From these, the first two modes are relevant when analyzing the gravity dam, however mode I is the most important, as in most cases a crack in the body of a gravity dam occurs due to excessive tensile stresses at the zones of weaknesses (for instance, at interfaces and joints within the body of the dam).

A series of expressions have previously been derived for obtaining the stresses and displacements in the vicinity of the crack tip (Irwin 1958): the values of stress intensity factors K are calculated using the Displacement and Stress Extrapolation Methods:

a) Displacement Extrapolation Method (plane strain condition):

$$K_I = \frac{E\sqrt{2\pi}}{4(1-\upsilon^2)}\left(\frac{u_2}{\sqrt{r}}\right)_{I\to 0} \tag{18}$$

Instead of obtaining an exact value of K_I at $r = 0$, the values of K_I are plotted against the distance r using Equation 18 and a straight-line extrapolation is used to find the value of K_I at the crack tip ($r = 0$).

b) Stress Extrapolation Method (plane strain condition):

$$K_I = \sqrt{2\pi}\left(\frac{\sigma_{22}}{\sqrt{r}}\right)_{r\to 0} \tag{19}$$

Again, instead of obtaining an exact value of K_I at $r = 0$, the values of K_I are plotted against the distance r using Equation 19 and a straight-line extrapolation is used to find the value of K_I at the crack tip ($r = 0$). For the present purposes, a parametric study is to be coducted in order to investigate the numerical stability and reliability of interface isoparametric quadratic singular finite elements and isoparametric quadratic singular boundary elements at a crack tip. Another major objective is to find the optimal sizes of interface isoparametric quadratic singular finite elements and isoparametric quadratic singular boundary elements in the vicinity of crack tips. To this end, the case of an elastic block with applied uniform shear stresses q_s at the top of the block is to be addressed. The assumed geometry, material properties of the block and applied loading, used in the analysis, are as follows: hight $H = 2.0$ m, width $L = 1$ m; $E = 106$ kN/m^3; $\nu = 0$; and $q_s = 50$ kPa. In total, about 400 computer runs were carried out covering different cases. These included use of different finite element—boundary element schemes. The analyses were carried out for different sizes of the crack in the body of the block. The crack was placed in the middle of the body, as well as at the base of the block.

The analysis started by placing a crack in the middle of the body of an elastic block. Stress intensity factor K_I was calculated by using 8-node isoparametric quadratic singular finite elements and interface isoparametric quadratic singular finite elements. Singular elements were placed at the crack from both sides of the crack tip. Finer mesh was employed near the crack with smaller elements placed in the vicinity of the crack tip. The values of normal and shear stiffness coefficients of interface elements were initially assumed to be $C_{nn} = C_{ss} = 10^8$ kN/m^3.

Initially it was assumed that the crack length was equal to $l = L/8$. The analyses were started with $a_1 = a_2 = l/2$ and at subsequent stages of the analyses the finite element mesh was refined by reducing the value of a_2 to $l/5$. The values of stress intensity factor K_I were calculated by means of using both, displacement and stress extrapolation methods. It should be noted that initially the value of K_I that was calculated by means of using the values of normal displacements u_y converged to those computed by means of using the values of vertical normal stresses σ_y. However, when the value of a_2 was equal to $l/5$, the values of K_I calculated by means of two alternative methods diverged from each other (Fig. 3). This may be due to low values of normal and shear stiffness coefficients of

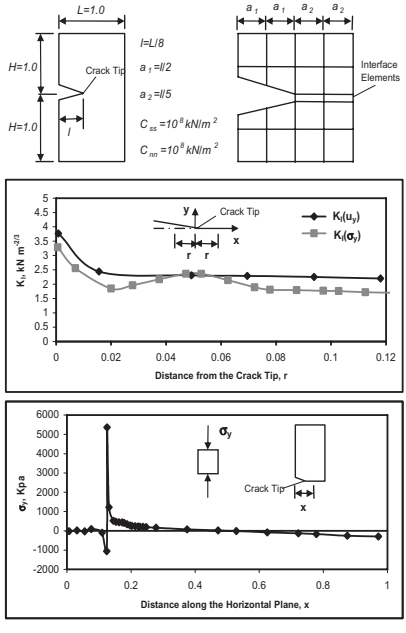

Figure 3. Calculation of stress intensity factor K_I (when a crack is placed in the middle of the body of an elastic block; $a_1 = l/2$ and $a_2 = l/5$).

interface elements that were assumed to be equal to $C_{nn} = C_{ss} = 10^8$ kN/m^3. These values are based on the parametric studies reported elsewhere (Raoof et al. 1998). Namely, it was found that the interface elements exhibited numerical stability with the values of normal and shear stiffness coefficients equal to $C_{nn} = C_{ss} = 10^8$ kN/m^3 when placed in the middle of the block. However, as higher stress gradients arise near a crack tip, these values may not be enough to model a good bond at the interface. Therefore, at the next stages of the analyses it was decided to increase the values of C_{nn} and C_{ss} up to 10^9 kN/m^3.

The values of stress intensity factor K_I calculated by means of using both, displacement and stress extrapolation methods, became closer to each other with $C_{nn} = C_{ss} = 10^9$ kN/m^3 and they converged to the same value when $a_1 = l/6$ and $a_2 = l/6$. These results are in close agreement with the results obtained by using only 8-node isoparametric quadratic singular finite elements.

At the next stage of the analysis a crack was placed at the base of an elastic block. The values of normal and shear stiffness coefficients of interface elements were assumed to be equal to $C_{\mathrm{nn}} = C_{\mathrm{ss}} = 10^9$ kN/m^3. It was assumed that the crack length was equal to $l = L/8$, where L is the width of the block. The analyses were started with $a_1 = a_2 = l/2$ and at subsequent stages of the analyses the finite element mesh was gradually refined with $a_1 = a_2 = l/3$, $l/4$, $l/5$, $l/6$, $l/8$, $l/10$ and $l/12$.. The values of stress intensity factor K_I were calculated by means of using both, displacement and stress extrapolation methods.

Again, despite a rather coarse mesh near the crack tip, the values of K_I calculated by means of using the values of normal displacements u_y exhibited much less scatter then those computed by means of using the values of vertical normal stresses σ_y. After refining gradually the finite element mesh near the crack tip, the values of stress intensity factor K_I converged to the same value when computed by using displacement and stress extrapolation methods.

It should be noted, that the necessary degree of the refinement of the finite element mesh to obtain the desired level of accuracy was higher as compared to the case when the crack was placed in the middle of the block. This can be due to higher stress gradients at the base of the block.

The parametric studies illustrated that the interface isoparametric quadratic singular finite elements can be effectively employed to model the problem of crack occurrence and propagation within the body of an elastic continuum and that the interface isoparametric quadratic singular finite elements can be effectively employed to model the problem of crack occurrence and propagation at the base of an elastic continuum.

6 EFFECT OF CYCLIC LOADING AND AGEING OF DAM CONCRETE

In what follows, a constitutive model for concrete in plane strain condition is modified in order to take into account the effect of material fatigue under slow static cyclic loading and the effect of ageing of concrete. Particular emphasis is placed on material response, simulating a realistic behavior of a dam under actual operating conditions.

This approach allows us to account for the effect of the degradation of concrete strength (fatigue) under cyclic loading and also for the effect of ageing of concrete. To this end, the value of uniaxial compressive strength of concrete σ_c in Equation 1 can be substituted by the value of the strength of concrete which has been appropriately modified in accordance with the number of loading/unloading cycles n and the time-period of the operation of a dam t:

$$\sigma_c = \sigma_c(n,t) \tag{20}$$

The value of strain ε_c associated with the maximum uniaxial compressive stress of concrete may also be modified in accordance with the number of loading/unloading cycles n and the time-period of the operation of a dam t:

$$\varepsilon_c = \varepsilon_c(n,t) \tag{21}$$

It is also well known that cyclic loading causes the degradation of concrete stiffness. At the same time, the passage of time causes the concrete modulus of elasticity to increase: these two effects are accounted for in the present model by means of modifying the value of the initial modulus of elasticity in accordance with the number of loading/unloading cycles n and the time-period of operation of a dam t:

$$E_0 = E_0(n,t) \tag{22}$$

Cycling loading exhibits significant nonlinear behaviour and drastic changes in material properties of concrete. The result is a considerable degradation of material properties of concrete as the number of applied loading/unloading cycles increases. In the present work, the empirical relationships based on the experimental findings on cyclic behaviour reported elsewhere (Osidze & Khoperia 1987) are adopted. Very briefly, following the tests carried out on the concrete specimens of Enguri arch dam, which were subjected to slow static cyclic compressive loading, the following relationships were established to define the degradation of the material properties of concrete in relation of loading/unloading cycles:

$$\begin{aligned} \sigma_c(n) &= \left(1 - a_\sigma^n \lg n\right)\sigma_c \\ E_0(n) &= \left(1 - a_E^n \lg n\right)E_0 \\ \varepsilon_c(n) &= \left(1 - a_\varepsilon^n \lg n\right)\varepsilon_c \end{aligned} \tag{23}$$

where parameters a_σ^n, a_E^n and a_ε^n define the degradation of the material properties of concrete under slow static cyclic loading; and n is the number of loading unloading cycles in accordance with the operation history of a gravity dam (it corresponds to the number of discharges/filling up cycles of the reservoir during the operation of the dam).

The specific values of these parameters may only be determined by carrying out cyclic tests on concrete specimens. Nevertheless, a careful study of the results of the investigations suggest that the values of the above parameters do vary within the following ranges:

$$\begin{aligned} 0.05 \le a_\sigma^n \le 0.25 \\ 0.10 \le a_E^n \le 0.30 \\ 0.10 \le a_\varepsilon^n \le 0.30 \end{aligned} \tag{24}$$

It is also interesting to note that, in general, the rate of degradation of material parameters and the strength of concrete under static cyclic loading, primarily depends on the level of stresses (i.e. the level of loading imposed on the specimen). For example, the value of modulus of elasticity for a concrete specimen was found to have been reduced by 51.5% (from 39780 MPa to 19300 MPa) after applying 150 loading/unloading cycles when the level of applied stresses was equal to $0.2\sigma_c$, with σ_c = the uniaxial compressive strength of concrete. When subjected to the same number of loading/unloading cycles (i.e. 150), with the level of applied stresses set equal to $0.5\sigma_c$ and $0.8\sigma_c$, the corresponding values of moduli of elasticity for the concrete specimen were found to have been reduced by 29.3% (from 33390 MPa to 23620 MPa), and 20.9% (from 28390 MPa to 22500 MPa), respectively (Osidze & Khoperia 1987). In the present work, average values of the above-defined coefficients (which seem to provide reasonable approximation of an actual behavior of concrete under cyclic loading) were adopted, although, it sould be noted that, in general, it is, indeed, possible to define the relationship between the parameters a_σ^n, a_E^n, a_ε^n and the level of stresses.

Another point, which needs a mention, is that the rate of degradation of material parameters and the strength of concrete under slow static cyclic loading also depends on the age of the test specimen. For example, the value of modulus of elasticity for a 28 days old concrete specimen was found to have been reduced by 51.5% (from 39780 MPa to 19300 MPa) after applying 150 loading/unloading cycles when the level of applied stresses was equal to $0.2\sigma_c$, while the value of modulus of elasticity for the corresponding 365 days (1 year) and 1825 days (five year) old concrete specimens (subjected to the same levels of applied stresses $0.2\sigma_c$) were found to have been reduced by 49.0% (from 39830 MPa to 21750 MPa) and 42.0% (from 42460 MPa to 20310 MPa), respectively, after applying the same number of loading/unloading cycles (i.e. 150) to each specimen. These figures, however, suggest that such differences are not practically significant and may reasonably be ignored.

The same approach can be used to describe the change in material properties of concrete due to ageing of the material. Again, the logarithmic function seems to predict well the variations of the material properties of concrete with time. Namely, the relationships can be expressed as:

$$\begin{aligned} \sigma_c(t) &= \left(1 + a_\sigma^t \lg t\right)\sigma_c \\ E_0(t) &= \left(1 + a_E^t \lg t\right)E_0 \\ \varepsilon_c(t) &= \left(1 + a_\varepsilon^t \lg t\right)\varepsilon_c \end{aligned} \tag{25}$$

where the parameters a_σ^t, a_E^t and a_ε^t define the modification of the material parameters of concrete due to ageing of concrete; and t is the number of years in accordance with the operation history of a gravity dam.

A careful study of the final results suggest that the values of the above parameters may vary in certain ranges for different types of concrete and specific values of these parameters have to be chosen from the results obtained from performing an in-situ observation on a concrete specimen. Namely, the values of the above coefficients vary in the following ranges:

$$\begin{aligned} 0.05 \le a_\sigma^t \le 0.15 \\ 0.05 \le a_E^t \le 0.15 \\ 0.05 \le a_\varepsilon^t \le 0.15 \end{aligned} \tag{26}$$

These parameters are based on the experimental findings (Osidze & Khoperia 1987). The authors observed the concrete specimens of Enguri arch dam. It can be seen that the values of the parameters a_σ^t, a_E^t and a_ε^t are generally lower than the values of the parameters a_σ^n, a_E^n and a_ε^n. This means that the effect of ageing of concrete on stress-strain curve of concrete is less significant than that of static cyclic loading. However, we have to mention here, that there may be different combinations of the number of years and the number of loading/unloading cycles applied during this period of time. Namely, if during the operation of a gravity dam, there is only one discharge/filling cycle of the reservoir during a year, the number of loading/unloading cycles will be equal to the number of years of the operation of the reservoir (i.e. $n = t$). However, if during the operation of a gravity dam, there are two discharge/filling up cycles of the reservoir during a year, the number of loading/unloading cycles will be twice as much as the number of years of the operation of the reservoir (i.e. $n = 2t$).

The above-defined values of the material properties of concrete are incorporated into the constitutive

equations defined in Paragraph 1 (constitutive model for concrete in the plane strain condition) in order to account for the fatigue of the concrete under cyclic loading and for the effect of ageing of concrete.

In our analyses the effect of ageing of concrete were investigated together with the effect of loading/unloading cycles. The objective of these analyses was to find out how the changes in the material properties of concrete due to ageing of concrete and subject to static cyclic loading affect the stress-strain curve of concrete under the plane stress and plane strain conditions. The analyses were performed only for compression-compression and compression-tension modes of stresses. The tension-tension mode of stresses can be assumed to be linear and it can also be assumed that the tensile strength of concrete varies proportionally with the changes in the value of compressive strength of concrete. In total, about 200 computer runs were made for different stress combinations α (which defines the value of the stress in the minor principal direction over the value of the stress in the major principal direction), for $t = 0$, 10, 25, 50, and 100, also for $n = t$ and $n = 2t$.

The analyses results for the compression—compression mode of stresses are presented in Figure 4. It should be noted that a significant

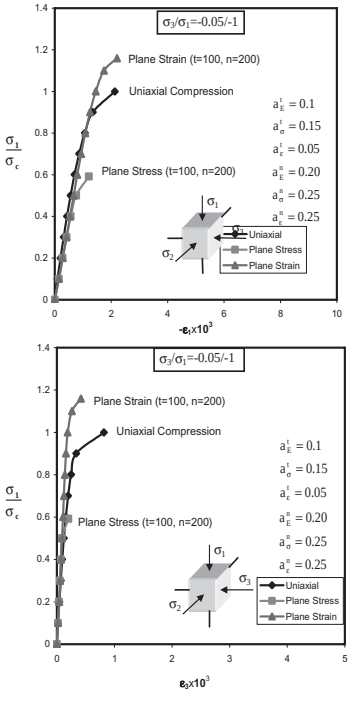

Figure 4. Comparison of uniaxial compression test, plane stress and plain strain conditions of concrete for compression-compression state accounting for the number of loading-unloading cycles n and age of concrete t with $\alpha = 0.05$.

degradation of the material parameters of concrete occurs even with $n = 10$. The degree of the degradation of the material properties of concrete increases together with the increase of the number of loading/unloading cycles n. The ageing of concrete has some favorable effect on the material properties of concrete, however the degradation of the material properties of concrete due to combined action of these 2 factors are still significant. Namely, when $\alpha = -0.15$ $t = 100$ and $n = 200$, the specimen will fail and a crack will occur in the plane strain condition when the compressive stress equals to only about 20% of the uniaxial compressive strength of concrete.

7 EFFECT OF CYCLIC LOADING AND AGEING OF CONCRETE ON INTERFACES

The shear stress-relative displacement curves obtained from shear tests subject to static cyclic loading are extremely rear. Here, we have to proceed in analogy to the previous paragraph, where the effect of cyclic loading on the material properties of concrete was numerically evaluated. It can be assumed that a large number of applied loading/unloading cycles will result in a degradation of the shear stiffness at interfaces.

The effect of cyclic loading on the shear stiffness k_s^0 can be accounted for by means of modifying the value of the initial shear stiffness associated with zero normal stress

$$\left(k_s^0\right)^{\sigma_n=0}.$$

In analogy to the previous paragraph, the following empirical relationship can be established to define the degradation of the value of the initial shear stiffness associated with zero normal stress

$$\left(k_s^0\right)^{\sigma_n=0}$$

in relation of loading/unloading cycles n:

$$\left(k_s^0\right)^{\sigma_n=0} = (1 - a_k^n \lg n)\left(k_s^0\right)^{\sigma_n=0} \tag{27}$$

where the parameter $a_k{}^n$ defines the degradation of initial shear stiffness associated with zero normal stress under static cyclic loading; n is the number of loading/unloading cycles in accordance with the operation history of a dam.

The degradation of the shear strength at interfaces under static cyclic loading can be accounted for by modifying the value of cohesion c in Equation 12. The degradation of the value of the shear strength in relation of loading/unloading cycles n can be defined by the following empirical relationship:

$$\tau_c = c(1 - a_\tau^n \lg n) + \sigma_n \tan\phi \tag{28}$$

where the parameter a_τ^n defines the degradation of the value of cohesion under static cyclic loading.

The values of the above coefficients vary in the following ranges:

$$0.10 \le a_\tau^n \le 0.25$$
$$0.15 \le a_\tau^n \le 0.30 \tag{29}$$

The effect of ageing of concrete is accounted for in a similar fashion. Namely, the initial value of the shear stiffness associated with zero normal stress, one has:

$$\left(k_s^0\right)^{\sigma_n=0} = (1 + a_k^t \lg t)\left(k_s^0\right)^{\sigma_n=0} \tag{30}$$

The effect of ageing of concrete on the shear strength at interfaces can be defined as:

$$\tau_c = c(1 + a_\tau^t \lg t) + \sigma_n \tan\phi \tag{31}$$

The values of the relevant coefficients vary in the following ranges:

$$0.05 \le a_\tau^t \le 0.15$$
$$0.10 \le a_\tau^t \le 0.20 \tag{32}$$

The one of the analyses results is shown in Figure 5, which depicts the effect of cyclic loading and the effect of ageing of concrete on the shear stress—relative displacement curve occurring at interfaces. It is clear, that the cyclic loading causes a significant degradation of material behavior of joints. Whenever available, in-situ monitoring data should be used for the analysis of a gravity dam and the above parameters should be modified in accordance with the actual behavior of concrete under a cyclic loading.

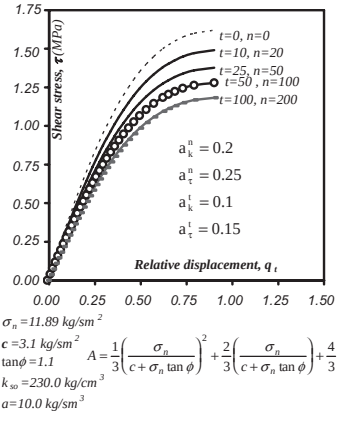

Figure 5. Shear stress/relative displacement curve taking into account the number of loading/unloading cycles *n* and the process of ageing of concrete *t* for interface between concrete and rock, with a variable value of parameter *A*.

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Modelización y análisis del comportamiento estructural de la presa de Mequinenza

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RESUMEN: El sistema de auscultación de la presa de Mequinenza detectó en su momento unos movimientos remanentes, muy acentuados en el bloque 13–14, que inicialmente se atribuyeron completamente a un hinchamiento de la masa del hormigón. Recientemente, se ha reformulado el diagnóstico y se ha planteado una nueva propuesta. A fin de contrastar las nuevas hipótesis de comportamiento, en esta comunicación se presentan resultados de una simulación numérica basada en el método de los elementos finitos 2D, que incluye elementos junta para representar la fisuración.

ABSTRACT: The auscultation system of Mequinenza dam has detected non-recoverable displacements, which were particularly pronounced in block 13–14. Initially, these displacements were completely attributed to the swelling of concrete. Recently, the diagnosis has been reformulated according with a new behavior hypothesis. To evaluate this new hypothesis, this paper present numerical simulation results based on 2D finite element models, which includes interface elements to represent cracking.

1 INTRODUCCIÓN

Por su importancia histórica y estratégica, la presa de Mequinenza es de vital importancia en el sistema hidrográfico español. Dispone desde sus primeras etapas en los años 60, de un complejo y completo sistema de auscultación, que han detectado en su momento unos movimientos remanentes cuya causa se atribuyó a un hinchamiento (entumecimiento) del hormigón de la presa. Las actuaciones realizadas durante la década de 1970 fueron eficientes para el control de los movimientos, si bien el diagnóstico dado presenta diversos aspectos cuestionables.

Recientemente, dentro de los trabajos usuales desarrollados entre la UPC y ENDESA en el estudio de presas con expansiones, se ha reformulado el diagnóstico y se ha planteado una nueva propuesta que explica de forma más consistente los movimientos producidos, y que en menor medida aún se producen (Buil et al., 2008).

A fin de contrastar la nueva hipótesis, en este trabajo se presentan resultados de una simulación numérica basada en el método de los elementos finitos (MEF) en 2D. La modelización incluye elementos junta sin espesor con un comportamiento constitutivo no lineal, que posibilita la simulación de la fisuración. El análisis incluye, además de las acciones debidas al peso propio y el empuje hidrostático, la consideración de las posibles expansiones y la acción de la subpresión con la apertura de juntas.

2 DESCRIPCIÓN DE LA PRESA Y COMPORTAMIENTO OBSERVADO

La presa de Mequinenza se integra en el aprovechamiento hidroeléctrico integral de un tramo del río Ebro gestionado por ENHER en 1955. Su construcción entre 1957 y 1964 dio origen a un embalse hiperanual de 1530 hm^3. Se trata de una presa de gravedad de planta recta, con altura máxima de 79 m, una longitud de coronación de 461 m y un volumen del cuerpo de presa de $1{,}1 \times 10^6$ m^3. En la figura 1 se presenta una vista de la presa desde la zona de aguas abajo. El perfil de la cerrada es abierto (tipo U), con una altura de cota muy similar en la mayoría de los bloques que por lo tanto presentan una rigidez similar. En la figura 1 se ha señalado el gran bloque donde se sitúan las tomas (junta 13 y 14), de una longitud de 70 metros. La longitud del resto de los bloques se sitúa entre 14 y 21 metros, excepto el bloque entre las juntas 21 y

Figura 1. Vista de la presa desde aguas abajo.

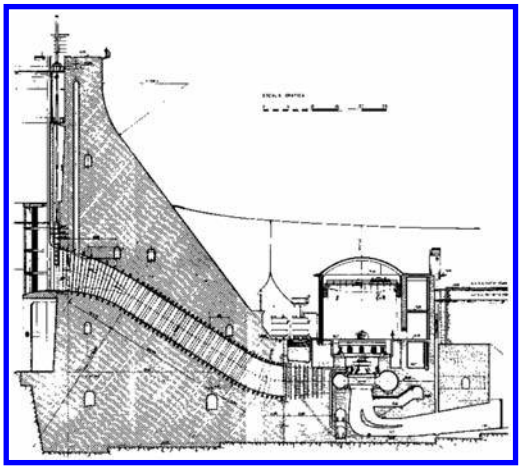

Figura 2. Sección transversal tipo del bloque 13–14.

22 que es de 8,16 m. En la figura 2 se muestra una sección transversal del bloque 13–14, por el que atraviesan las tuberías que van a las turbinas.

La presa se embalsó por primera vez en diciembre de 1965, si bien con posteridad se bajó de cota para volverse a llenar en 1969. Durante ese período se llevaron a cabo una serie de trabajos complementarios, entre los que se encuentran la ampliación de la pantalla de impermeabilización y drenaje, consolidación de las laderas, prolongación de las galerías de inspección y drenaje, construcción de una nueva galería profunda de drenaje a 25 m por debajo de la cimentación de la presa (Díez-Cascón y Bueno Hernández, 2001) y un rastrillo de refuerzo de la presa (Álvarez y Baztán, 1967).

Entre 1968 y 1971 se realizaron campañas de inyecciones, centradas principalmente en el bloque 13–14, a fin de rellenar grietas horizontales y atajar una filtración aparecida aguas abajo en la cota 112,75 m. Entre 1972 y 1974 se ejecutó la pantalla de drenaje entre coronación y la galería de la cota 100, y desde ésta hasta la galería de la cota 81.

Al comenzar el llenado del embalse, durante los primeros años pudo observarse unos movimientos de las alineaciones hacia aguas abajo, más acentuados en las cinco bases del bloque situado entre las juntas 13 y 14. En la figura 3 se presentan los movimientos de las alineaciones de las bases de coronación situadas en los distintos bloques, para cuatro edades diferentes: 1979, 1986, 1996 y 2006. En ella puede verse, por un lado, que los movimientos principales de la presa se produjeron en los primeros años, ya que en el año 1979 ya se habían producido la mayor parte de los mismos. Sin embargo, no se puede señalar que el proceso haya finalizado, si bien los movimientos posteriores son pequeños comparados con los correspondientes a los primeros años

Por otro lado, puede apreciarse el comportamiento diferenciado de las bases situadas en el bloque

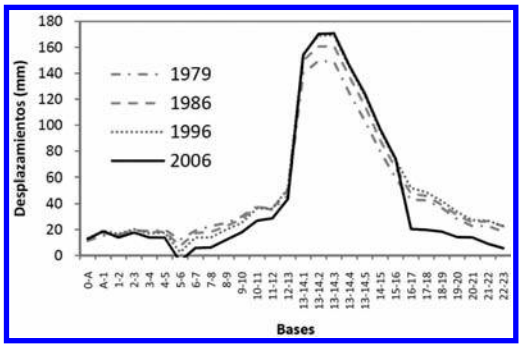

Figura 3. Movimientos de alineación en coronación de la presa.

13–14, en el que se producen unos desplazamientos de las alineaciones de una magnitud considerablemente mayor, si bien se observa cierto movimiento de arrastre en los bloques vecinos, sobre todo el 14–15 (puede apreciarse que este efecto de arrastre es bastante inferior en el bloque vecino 12–13, a partir de donde se encuentran los bloques que contienen el aliviadero). La figura permite detectar la superposición de dos componentes del movimiento, uno general para el conjunto de la presa, y otro más acentuado concentrado en el bloque 13–14.

En la figura 4 se muestra la evolución en el tiempo de los movimientos de las alineaciones en la base 13–14.4 a diferentes alturas (en coronación y en galerías ubicadas aproximadamente a cota 100, 81 y 51 m). En ella puede verse que, aparte de las variaciones estacionales, se presentan unos movimientos remanentes, más significativos cuanto mayor es la cota de la base de medida. También se

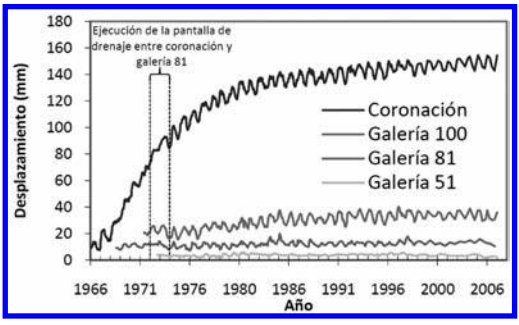

Figura 4. Evolución en el tiempo de los movimientos de alineación en la base 13–14.4.

observa en la base de coronación un crecimiento más acentuado de los movimientos en los primeros años, que posteriormente decrece y mantiene una pequeña pendiente no nula. En dicha figura se ha señalado el período en que se ejecutaron los drenes entre coronación y la galería 81.

Por su parte, en las nivelaciones se observa un comportamiento similar al de las alineaciones, en cuanto a la singularidad del bloque 13–14 y un crecimiento más acentuado en los primeros años, si bien la magnitud de los movimientos registrados resulta menor.

3 HIPÓTESIS SOBRE LAS CAUSAS DEL COMPORTAMIENTO OBSERVADO

Los técnicos de la propiedad en aquel momento, tras descartar diversas causas, encargaron estudios experimentales para analizar la posible expansión del hormigón y el comportamiento inusual del bloque 13–14 (Fernández Cánovas, 1980). En los ensayos realizados por Fernández Cánovas en la segunda mitad de la década de los años 1970 (con testigos de 5,6 cm de diámetro extraídos de la presa, secados en estufa a 100°C hasta peso constante y sumergidos en agua sin presión a 20°C), se obtuvieron valores máximos de expansión después de los 140 días de hasta 0,7 mm/m.

Los ensayos experimentales han sido un condicionante para el desarrollo posterior de los trabajos de análisis estructural del comportamiento de la presa. Ahora bien, cabe cuestionar esos resultados ya que la técnica experimental empleada no es representativa de la situación real de la presa. Los valores reales de entumecimiento del hormigón de la presa en zonas totalmente saturadas deben ser significativamente inferiores a los obtenidos en los ensayos. Además, dado que no hay diferencias significativas de hormigón y de sus componentes en el bloque 13–14 con respecto al resto de bloques de la presa, cabría esperar entonces que los movimientos en los otros bloques reflejaran el mismo comportamiento en las alineaciones.

Por otro lado, en los estudios experimentales mencionados se ha descartado una posible expansión del hormigón por reacciones químicas, como causa de la anomalía detectada.

Con posterioridad (finales de la década de los años 70), se hizo una evaluación estructural con el MEF, mediante el programa Strudel, considerando entre las acciones el entumecimiento. Años después, en la década de los 80, se realizaron unos estudios numéricos, con el entumecimiento como acción principal, con objeto de intentar ajustar los resultados medidos en la presa y los obtenidos mediante métodos numéricos más avanzados (Oliver et al., 1985 y Cervera et al., 1990). Estos trabajos no cuestionan el entumecimiento como causa principal, si bien en algunos resultados tienen que adoptar valores elevados del mismo (incluso superiores a los experimentales).

El planteamiento anterior presenta varios puntos débiles como la consideración de valores excesivos de entumecimiento, técnica experimental poco representativa, efecto de entumecimiento demasiado diferenciado entre el bloque 13–14 y los restantes, etc. En trabajos más recientes (Buil et al., 2008 y Buil et al., 2009) se ha retomado el estudio global de la presa y se ha reformulado el diagnóstico con una nueva visión de la principal causa del comportamiento diferenciado del bloque 13–14. La nueva hipótesis incluye dos efectos intervinientes, uno que se produce en el conjunto de la presa y actúa en todos los bloque, y otro que afecta al bloque 13–14, con un consecuente arrastre en bloques adyacentes, tal como puede verse en la figura 3 presentada anteriormente.

El efecto generalizado sí se produciría como consecuencia del entumecimiento del hormigón, pero con valores de expansión mucho más reducidos y uniformes en todos los bloques. Por su parte, el efecto localizado en el bloque 13–14 se habría producido como consecuencia de la apertura de fisuras ubicadas en la zona superior de paramento aguas arriba (grietas situadas en juntas de construcción) durante los primeros años de explotación de la presa. Debido a las características singulares de este bloque (mayores dimensiones y existencia de tuberías forzadas en su interior), las juntas de hormigonado se han visto debilitadas por efectos térmicos y de retracción, lo cual ha favorecido la aparición, evolución y apertura de las mismas, con la consiguiente actuación de la subpresión.

El nuevo diagnóstico también incluye la posibilidad de otros efectos debidos, por un lado, a una expansión de lignitos bajo presión de agua (presentes en la cimentación en estratos de pequeña potencia), y por otro lado, no se descartan posibles expansiones químicas diferidas, si bien en ambos casos con un reducido aporte.

Con la finalidad de contrastar las principales hipótesis formuladas en la nueva propuesta de diagnóstico del comportamiento de la presa de Mequinenza, en el siguiente apartado se presentan diversos resultados de una simulación numérica basada en el método de los elementos finitos en 2D, que incluye elementos junta para representar la fisuración.

4 ANÁLISIS NUMÉRICO DE UNA SECCIÓN TRANSVERSAL

4.1 *Modelización*

Para la representación geométrica de la sección transversal del bloque 13–14, se ha partido de la información y planos facilitados por ENDESA S.A. Se han incorporado en el modelo planos potenciales de fisuración, para lo cual se ha tenido en cuenta un levantamiento detallado de fisuras realizado en paramento aguas arriba de la presa a finales de los años 60.

En la figura 5 se muestra la geometría representada en la modelización, que incluye la tubería de captación. El hormigón está representado en color azul oscuro, el terreno en verde, la tubería de captación en azul cielo, la junta de contacto presa-terreno en amarillo y las juntas horizontales de construcción en color rojo.

En la figura 6 se presenta la malla generada (detalle del bloque de hormigón), en la cual se puede apreciar una mayor densificación en la parte superior alrededor de la zona de ubicación de las juntas.

En el análisis numérico de problemas ingenieriles mediante el método de los elementos finitos (MEF), con frecuencia resulta conveniente el empleo de elementos junta sin espesor a fin de representar adecuadamente las discontinuidades y la fisuración. El comportamiento de la junta está formulado en términos de las componentes normal y tangencial de las tensiones en el plano de la misma y los correspondientes desplazamientos relativos. Las leyes constitutivas de estos elementos junta permiten un acoplamiento de los efectos normal/corte. El modelo constitutivo utilizado en este trabajo, está basado en la teoría de la elastoplasticidad e incorpora conceptos de la Mecánica de Fractura (Carol et al., 1997; López, 1999, López et al., 2008).

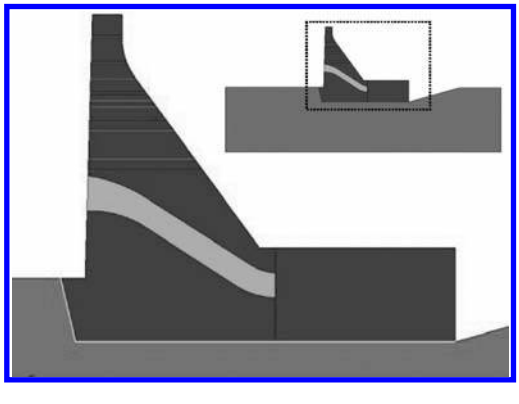

Figura 5. Geometría del perfil transversal del bloque 13–14.

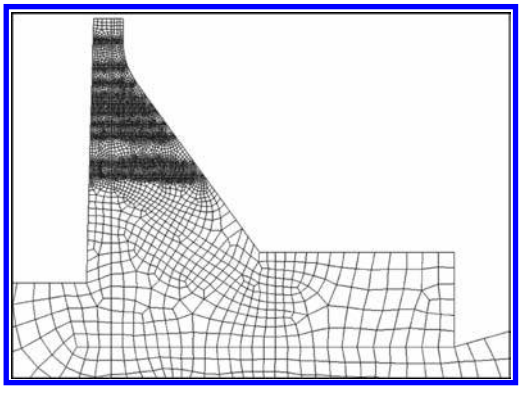

Figura 6. Detalle de la malla de elementos finitos.

4.2 *Resultados*

Para los cálculos se ha adoptado un módulo de elasticidad de 20 GPa para el hormigón y de 45 GPa para el terreno. La tubería se ha considerado como un hormigón equivalente de 13,33 GPa (su módulo corresponde a la proporción de la longitud total de hormigón dividido por la longitud total del bloque en la sección longitudinal en que se encuentra la tubería). El coeficiente de Poisson $\nu = 0{,}2$ para los tres materiales.

La junta de contacto presa-terreno se ha considerado elástica lineal con constantes de rigidez normal y tangencial de valor muy elevado (muy solidaria). Para las juntas de construcción representadas explícitamente se utiliza la ley de comportamiento no lineal, que posibilita la formación y apertura de las fisuras en las juntas.

Para generar la geometría y malla de elementos finitos, así como para la posterior representación gráfica de los resultados del cálculo, se ha utilizado el programa GID desarrollado en el CIMNE, UPC. Por su parte, los cálculos se han llevado a cabo con el código de elementos finitos denominado DRAC, desarrollado en el Departamento de Ingeniería del Terreno, Cartográfica y Geofísica de la ETSECCPB-UPC.

4.2.1 *Resultados iniciales*

En primer lugar, se evaluaron los efectos de las acciones del peso propio y del empuje hidrostático. El desplazamiento calculado en coronación para el punto superior de la línea vertical que pasa por las galerías, fue de 8 mm en horizontal hacia aguas abajo y 1 mm en dirección vertical. El valor calculado de las tensiones ha resultado muy bajo, como era de esperar.

A fin de tener en cuenta la incidencia de un proceso expansivo debido al entumecimiento, se plantearon diversos casos de estudio considerando la acción de una expansión en la zona de hormigón cercana a paramento aguas arriba y hasta aproximadamente la línea de drenes (unos 6,1 m de penetración), como se puede ver en la figura 7. Mediante un análisis elástico-lineal, se determinó la deformación impuesta necesaria para producir unos movimientos en coronación del mismo orden de magnitud que los registrados en las bases de medida de la presa. Como resultado se ha obtenido que se requiere una deformación de aproximadamente 1 mm/m para generar un movimiento de 15 cm en horizontal y 5,75 cm en vertical (medidos en coronación en la línea vertical de las galerías). Esta deformación resulta demasiado alta para adjudicarla a efectos del entumecimiento, y además diversos estudios sobre testigos de la presa descartan la posibilidad de otras posibles expansiones por procesos reactivos internos. A fin de acotar el aporte en los movimientos de un grado de expansión razonable por efectos de entumecimiento, para un cálculo elástico-lineal con una deformación constante reducida a 0,35 mm/m, se han obtenido unos desplazamientos en coronación de 5,96 cm en horizontal y 2,18 cm en elevación.

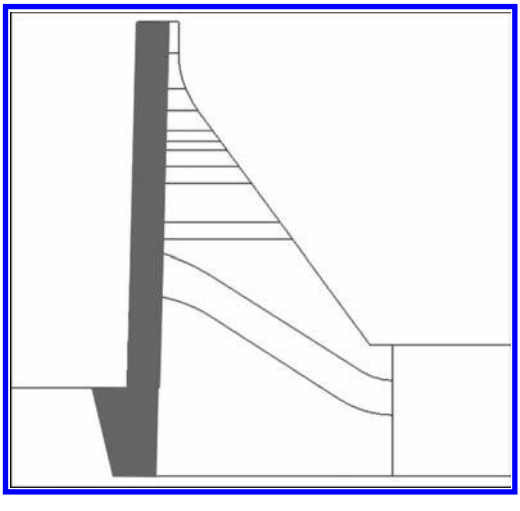

Figura 7. Zona en que se han aplicado las deformaciones impuestas (en color rojo).

4.2.2 *Análisis de la fisuración en el bloque 13–14*

Se aplica un valor de deformación impuesta de 0,35 mm/m, considerando ahora un comportamiento constitutivo no lineal de los elementos junta, y por lo tanto posibilitando la formación y aperturas de fisuras en las juntas de construcción, que se han representado explícitamente en la modelización. También se tiene en cuenta la acción de la subpresión. La resistencia a tracción de las juntas se ha estimado en un valor bajo de 0,5 MPa, fundamentado en la hipótesis de la singularidad del bloque 13–14, en que debido a tensiones internas causadas por efectos térmicos y de retracción (se trata de un bloque de mayor tamaño que los demás y en el que probablemente el hormigón tiene un mayor contenido de cemento), las habituales juntas de hormigonado de una presa, en el caso de este bloque presumiblemente han resultado bastante débiles.

A fin de tener en cuenta un proceso más realista del entumecimiento, las deformaciones impuestas se han aplicado suponiendo una evolución que se inicia en paramento aguas arriba y progresa hasta llegar a una profundidad de 6,1 m (ver figura 7). La acción de la subpresión se ha ido incluyendo en el cálculo a medida que se produce la fisuración en cada junta. A partir de que se alcanza la resistencia máxima y comienza la degradación y apertura de una junta, empieza a actuar la subpresión, incluida en el cálculo mediante cargas distribuidas de compresión aplicadas sobre los elementos de medio continuo ubicados a uno y otro lado de la junta que se ha abierto.

En la figura 8 se presenta la evolución de la fisuración hasta alcanzar el estado final de la simulación numérica, en términos de la energía disipada durante el proceso de fractura, representada por el espesor de línea en cada punto de las juntas, en color rojo si la fisura está activa y en azul si se encuentra en descarga elástica.

La acción expansiva genera estados de compresión en la zona que actúa y tracciones en la zona vecina al frente de avance. En esta simulación la fisuración comienza a cierta distancia del paramento aguas arriba (figura 8a) y con la evolución de la expansión se produce un proceso de fisuración progresiva hacia aguas abajo, y también hacia el paramento aguas arriba (figuras 8b, 8c y 8d). En el proceso de evolución de la fisuración progresiva algunas zonas entran en descarga. Se puede observar que en estados avanzados de carga se alcanza longitudes considerables de propagación de la fisuración (cabe señalar apreciaciones en este sentido en la propia presa, por ejemplo en un informe interno del año 1969 se escribe que en el bloque 13–14 "una filtración en el paramento aguas abajo a la cota 112, 10 indica que se forma una grieta horizontal de paramento a paramento").

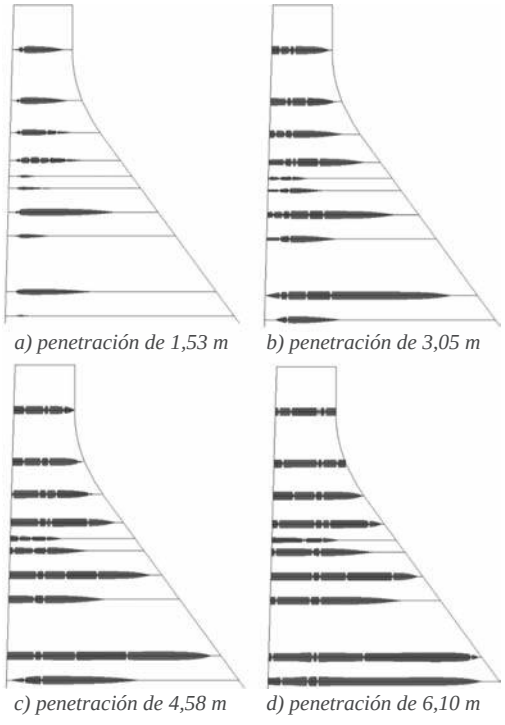

Figura 8. Evolución del estado de fisuración en términos de la energía disipada, en color rojo en carga y en azul en descarga.

En la figura 9 se muestra la configuración deformada final. Se ha obtenido un movimiento en coronación de 10,81 cm en horizontal y de 5,59 cm en elevación. La máxima apertura de fisura obtenida en este cálculo ha sido de 0,77 cm registrada en la junta superior.

Con el objetivo de evaluar la influencia de la subpresión, se ha repetido el cálculo sin incluir su acción en las juntas de construcción. Como resultado se ha obtenido un grado de fisuración similar, si bien con desplazamientos en coronación algo menores, con valores de 9,05 cm en horizontal y 4,45 cm en elevación.

4.2.3 *Aumento de la resistencia en las juntas*

A fin de contrastar la hipótesis de un comportamiento mecánico diferenciado en el bloque 13–14, se plantea ahora un caso de carga igual al anterior, pero considerando unas condiciones de mayor resistencia en las juntas de construcción, tal como cabe esperar en otros bloques vecinos al 13–14, que presentan una longitud considerablemente menor aunque una similar sección transversal. Para tener en cuenta este aspecto sólo se ha cambiado la resistencia a tracción de las juntas, que se ha adoptado de 3 MPa.

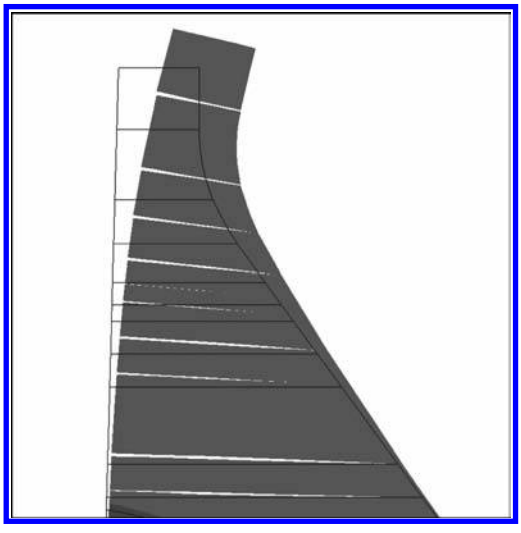

Figura 9. Configuración deformada final en la zona superior del bloque (×100).

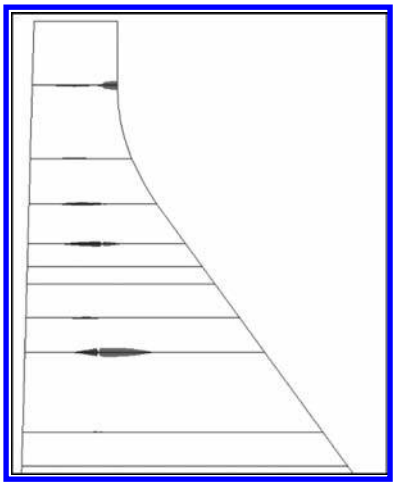

Figura 10. Estado final de fisuración en términos de la energía disipada (rojo en carga y azul en descarga) para una resistencia de 3 MPa en las juntas.

En la figura 10 se presenta el estado final de fisuración. Se puede apreciar en la figura que la simulación de un aumento de resistencia en las juntas de construcción, ha motivado una considerable reducción de la fisuración. Como consecuencia de esto, se ha obtenido una reducción de aproximadamente la mitad en los desplazamientos en coronación, que han sido de 5,82 cm en horizontal y de 2,47 cm en elevación, muy similares a los obtenidos en el cálculo elástico-lineal (5,96 y 2,18 cm, respectivamente).

4.2.4 *Influencia del nivel de expansión*

A fin de evaluar la influencia del nivel de expansión en los desplazamientos, se han realizado cálculos con otros dos diferentes valores de expansión, uno de 0,20 mm/m y el otro de 0,45 mm/m (ambos con resistencia de juntas de 0,5 MPa).

En las figuras 11 y 12 se presentan los resultados numéricos obtenidos en coronación para los desplazamientos horizontales y verticales, respectivamente, en función de diferentes niveles de expansión. Junto con los resultados del cálculo no lineal que permite la apertura de las juntas de hormigonado, se muestra la recta que representa el comportamiento con juntas elásticas (movimientos debidos sólo a las deformaciones del medio continuo). Se puede apreciar claramente en la figura el rápido crecimiento de los desplazamientos con el aumento de las deformaciones impuestas que representan el grado de entumecimiento. Cómo se ha visto, esto ocurre como consecuencia de la formación, evolución y apertura de fisuras en las juntas de construcción, que se han considerado más débiles en el bloque 13–14. El proceso se acentúa, con un crecimiento no lineal, a medida que aumenta el nivel de la deformación impuesta, lo cual permite alcanzar valores de los

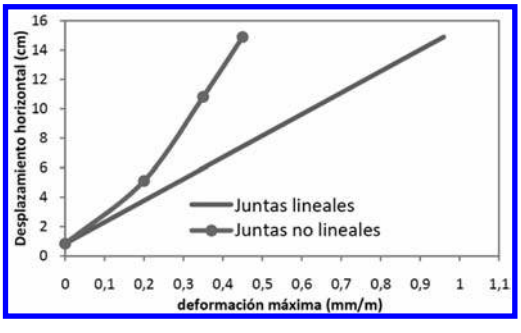

Figura 11. Desplazamiento horizontal en coronación en función del valor de deformación impuesta.

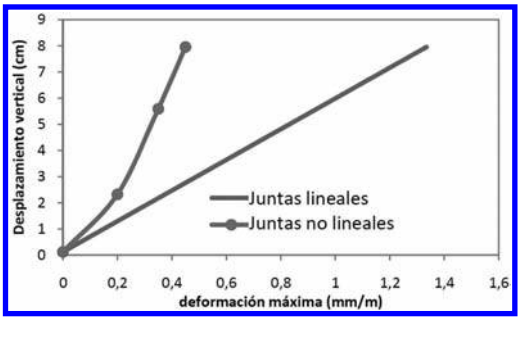

Figura 12. Desplazamiento vertical en coronación en función del valor de deformación impuesta.

desplazamientos similares a los medidos en este bloque, dentro de un rango más razonable de expansiones debidas a un entumecimiento (valores comprendidos entre 0,35 y 0,45 mm/m).

5 CONCLUSIONES

En este trabajo se ha presentado una simulación numérica basada en el método de los elementos finitos 2D, que incluye elementos junta para representar la fisuración. Se han incorporado en el modelo planos potenciales de fisuración, coincidiendo con algunas juntas de construcción, para lo cual se ha tenido en cuenta un levantamiento detallado de fisuras realizado en paramento aguas arriba de la presa.

Los resultados numéricos obtenidos permiten explicar de forma más consistente el comportamiento diferenciado detectado en la presa de Mequinenza, sin tener que recurrir a unas expansiones diferenciadas entre los bloques, y sobre todo sin necesidad de dar una magnitud excesiva de entumecimiento en el bloque 13–14, a fines de ajustar los movimientos medidos en la presa.

Un aspecto clave de la modelización ha sido el empleo de elementos junta. La acción combinada de una expansión debida a un entumecimiento que va avanzando desde paramento aguas arriba, junto con la consideración de planos de debilidad en el bloque 13–14, ha posibilitado la simulación numérica de formación y posterior evolución de fisuras (o grietas). La formulación de los elementos junta en términos de desplazamientos relativos, permite representar unas aperturas de fisura que contribuyen decisivamente en el aumento de los desplazamientos de este bloque. En cambio, una simulación sólo con medio continuo, requiere unos valores de expansión muy altos y poco realistas. Para la simulación adoptada, niveles expansivos comprendidos entre 0,35 y 0,45 mm/m permiten obtener valores de los desplazamientos horizontales y verticales en coronación similares a los registrados en el bloque 13–14 (quizás tendiendo a 0,35 si se incluyeran otras posibles fisuras no consideradas en la estimación).

Por otro lado, adoptando una mayor resistencia en las juntas de construcción tal como cabe esperar en otros bloques vecinos al 13–14, la situación cambia totalmente ya que la simulación muestra que se reduciría considerablemente la fisuración, y en consecuencia la magnitud de los movimientos.

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Se agradece la colaboración de Endesa Generación S.A en el desarrollo del estudio conjunto con la

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Cracking pattern evaluation on an arch dam by means of numerical modeling

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ABSTRACT: This article evaluates the thermal and mechanical behavior of a thin arch dam using finite difference based numerical models to reproduce the construction sequence. The analysis takes hydration heat generation and its diffusion through the structure into account. The stress and temperature distribution is estimated and its evolution as the dam is constructed is modeled. An attempt is made to explain the existing crack pattern in the dam.

1 INTRODUCTION

Theme A of the 10th Benchmark Workshop on Numerical Analysis of Dams (ICOLD) consist in the evaluation, for an arch dam, of the initial stress-strain pattern and the development of short-term thermal and rheological effects, considering a realistic construction sequence. In particular, for thin arch dams, these effects can become critical as they can be responsible of high tensile stresses and cracks, thus potentially triggering the safety of the structure.

Once the concrete is poured as scheduled through the construction process, the hardening phase starts due to cement chemical reactions which control the thermo-mechanical behaviour of fresh concrete. Besides, these reactions are influenced by hydration heat and its diffusion causing the thermal stress development inside the dam body and, potentially, cracking phenomena.

This type of analysis has to be performed by means of numerical modelling, being impossible to provide a complete analytical solution.

For the solution presented in this paper, the problem has been fully solved by FLAC 3D (Itasca), while SAP2000 NL (Berkeley) has been used to check a simplified mechanical model. The reason of making such simplified mechanical model (without considering the construction sequence) in order to check the order of magnitude of the stresses in the dam using SAP2000 NL, was due to the lack of experience of the working team with FLAC 3D.

FLAC 3D is a finite differences program developed by Itasca Consulting Group, Minnesota. This software uses the finite differences method to analyze the thermo-mechanical behaviour of a continuous medium in 3D until it reaches an equilibrium state.

SAP2000 NL is a finite elements program developed in Berkeley University, California. This software uses the finite elements method to solve any type of problem concerning structural analysis.

2 STATEMENT OF THE PROBLEM

The dam is a symmetric thin arch dam which main features are given in the next Figure.

The dam has been built in 7 months, from April to October, while the sealing of the joints in 2 months from December to January, following the construction. During the casting sequence each block was built in contact to the adjacent ones and the sealing of the joints occurred at the

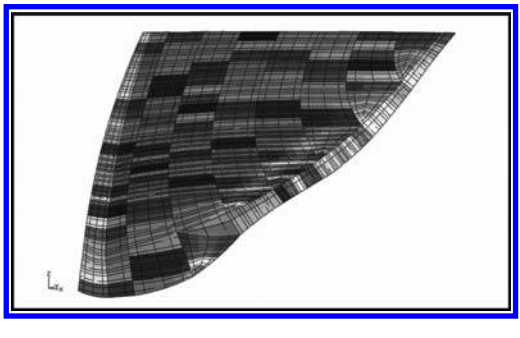

Figure 1. Main features of the dam.

end of construction. Therefore, the dam could be considered as continuous and monolithic with reference to the evaluation of the thermal field and to determine the stress-strain state. Nevertheless, such assumption should be properly justified or suitably integrated in the model (e.g. including interfaces).

Participants must justify their calculation assumptions and provide a possible explanation of the cracks pattern qualitatively given by the formulators. In addition, for the specified locations, participants should provide the the time histories of:

- Internal temperatures (°C, ASCII format)
- Stress tensor (MPa, ASCII format) up to the end of construction and optionally, if construction joints are modelled, up to the end of the joint sealing sequence.

The next Figures show the location of the joints where stress results are to be provided and the crack pattern that should be explained by the participants.

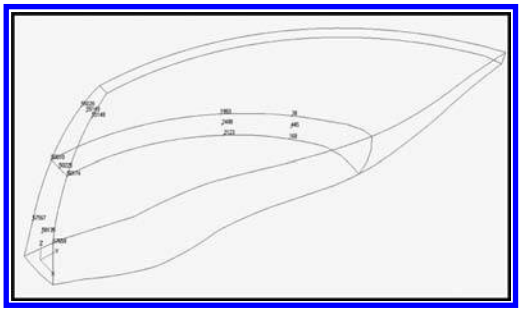

Figure 2. Location of the joints where stress results are to be provided.

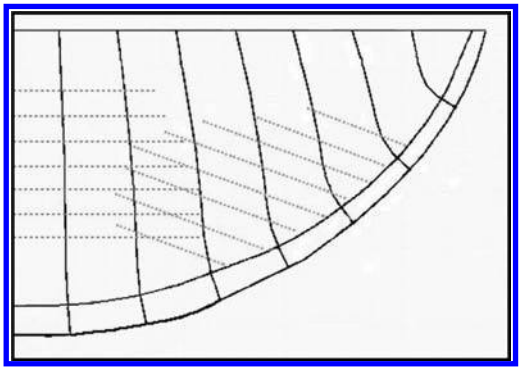

Figure 3. Crack pattern to be explained.

3 FEATURES OF THE IMPLEMENTED MODELS

3.1 *Geometrical models*

From the files provided by formulators (3DRefined-Model.uns), all needed information was extracted to create an excel file, that could later be imported by FLAC 3D and SAP2000.

This file (Modelo.xls) contained the coordinates of the nodes, the groups of nodes that are part of the solid elements, and the boundary conditions including some particularities for the two software packages.

3.1.1 *SAP2000 NL*

Since SAP defines its 3D elements as solid formed with eight nodes and six plane faces, geometrical information has been adapted to ensure that solids are properly defined.

Next Figure shows the geometrical model of the dam (formed by 40696 nodes and 8236 solid elements).

3.1.2 *FLAC 3D*

Since FLAC 3D defines its 3D elements as bricks formed with eight nodes and six plane faces, geometrical information was also adapted to ensure that solids are properly defined.

The next Figure shows the FLAC 3D geometrical model of the dam (formed also by 40696 nodes and 8236 solid elements):

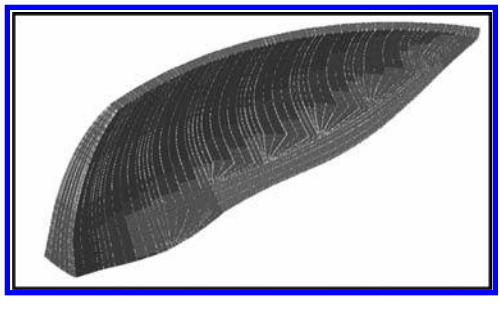

Figure 4. SAP2000 NL geometrical model.

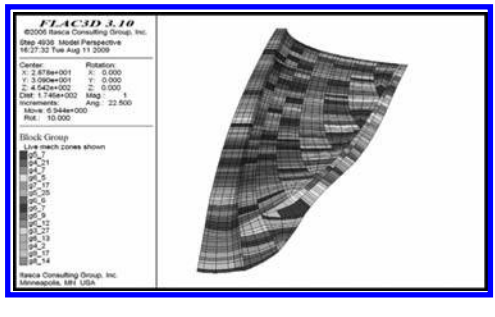

Figure 5. FLAC 3D geometrical model.

3.2 *Mechanical restraints*

Nodes that define the contact dam-foundation (bottom part of the dam) have been considered restrained in all directions. Nodes belonging to the symmetry plane have restrained in the Y-axis (perpendicular to the symmetry surface).

3.3 *Thermal/mechanical model*

As SAP2000NL has been applied only under mechanical static conditions to check how it compares to FLAC 3D, all considerations given in this section refer to the coupled thermo-mechanical transient model developed in FLAC 3D.

Starting by the thermal computations, the process is described by the following differential equation which is solved inside de software:

$$\rho \cdot C \cdot \frac{\partial T}{\partial t} - k \cdot \left(\frac{\partial^2 T}{\partial x^2} + \frac{\partial^2 T}{\partial y^2} + \frac{\partial^2 T}{\partial z^2} \right) = q_v \qquad (1)$$

where ρ is the density, C is the specific heat, k is the conductivity and q_v is the hydration heat generation.

In coupled thermal-mechanical modelling, solution of thermal-stress requires reformulation of the incremental stress-strain relations, which is accomplished in FLAC 3D by subtracting the portion due to temperature change from the total strain increment. The mechanical model selected for the solution herein presented is linear-elastic.

Because free thermal expansion results in no angular distortion in an isotropic material, the shearing-strain increments are unaffected. The thermal-strain increments associated with the free expansion corresponding to temperature increment ΔT have the form:

$$\Delta\varepsilon_{ij} = \alpha_t \, T \, \delta_{ij} \qquad (2)$$

where α_t [1/°C] is the coefficient of linear thermal expansion, and δ_{ij} is the Kronecker delta.

The differential equation of motion and the rate of strain-velocity relations are based upon mechanical considerations, and are unchanged for thermomechanics.

Detailed information on how the different factors affecting the thermo-mechanical behaviour of the dam have been addressed is given in the following sub-sections.

3.3.1 *Environmental conditions*

Data provided by the formulators consists in the external temperature time series. The external temperature affects all the thermal processes that are developed during dam's lifetime. In the early years, this temperature controls how the heat is transmitted from inside the dam to the exterior. This phenomenon occurs because the concrete generates a heat flux while it is hardening. After that, a cyclic process of heat diffusion which fluctuates depending on the outside temperature variations will affect the dam during all its life.

In this analysis, according to the information given, the function used to define the external temperature variation is:

$$T_{ext}(t) = 8.5 + 9 \cdot \sin\left(\frac{2 \cdot \pi \cdot t}{365} \right) \qquad (3)$$

where "t" is expressed in days.

The next Figure shows the variation during the construction period.

3.3.2 *Thermo-mechanical properties of concrete*

A series of values with regard to a number of thermo-mechanical constants have been provided by the formulators and considered in the analysis. Some of these properties vary with time, in particular Young's modulus and the hydration-heat generation (this second function will be properly defined in next sub-section).

The following Table provides the values and units that have been adopted for calculation.

The Young's modulus is defined by the following function:

$$E(t) = 28000 \cdot e^{\left(-\frac{0.47}{t} \right)^{1.08}} \qquad (4)$$

where t is expressed in days and E in MPa.

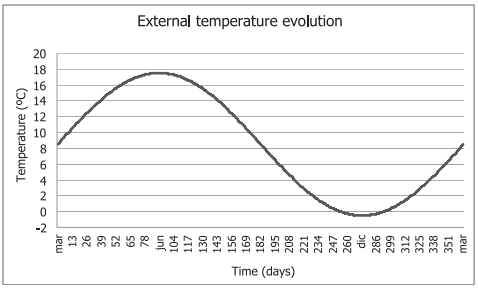

Figure 6. External temperature values considered in the analysis.

Table 1. Thermo-mechanical constants.

Weight density	2400 Kg/m^3
Specific heat	879 J/(kg °C)
Conductivity	2.5 W/(m °C)
Heat transfer coefficient	16 W/(m^2 °C)
Poisson coefficient	0.15
Coefficient of thermal expansion	10^{-5} °C^{-1}

Compressive and tensile strength are obtained through the empirical expressions given, where the considered constants have the following values: $K_c = 3.539 \cdot 10^{-8}$ and $K_t = 2.455 \cdot 10^{-9}$.

Therefore, the expressions are:

$$f_C(t) = 3.539 \cdot 10^{-8} \cdot E(t) \quad (5)$$

$$f_t(t) = 2.455 \cdot 10^{-9} \cdot E(t) \quad (6)$$

Finally, at the end of the construction (infinite time) the strength values that should be considered for the comparison are given in the following table.

3.3.3 *Construction issues and thermal boundary conditions*

As mentioned in several previous sections, the most relevant factor during the dam construction phase and its earliest years until concrete has completely hardened is the hydration-heat generation. The reactions that take place while concrete is curing are exothermic and it generates an internal heat known as hydration heat.

The following Figure, provided by the formulators, shows the construction schedule for all dam zones. In order to reproduce that process, which is extremely important to evaluate how hydration heat is developed in the dam body, another excel file (Modelo_automatización_zonas.xls) was created where each solid element was linked to one particular construction date.

Formulators have also provided the accumulated hydration heat at time "*t*", which is defined by the function:

$$q = \frac{t}{0.0653 + 0.0175 \cdot t} \quad (7)$$

where t is expressed in days and q in $Kcal/kg_{cement}$.

Table 2. Compressive and tensile strength.

Compressive strength	27.75 MPa
Tensile strength	1.92 MPa

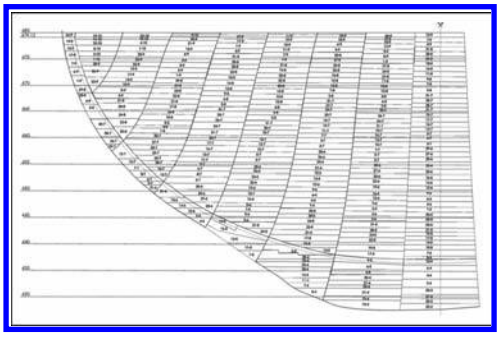

Figure 7. Construction sequence.

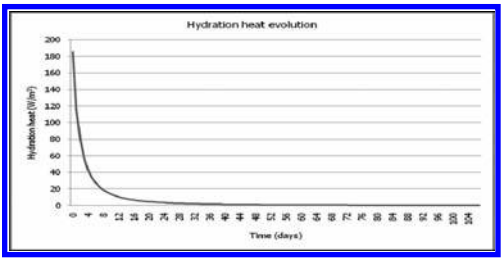

Figure 8. Hydration heat evolution.

In order to reproduce such phenomenon inside the FLAC 3D model, all volume of concrete constructed at a particular time is linked to a heat "source", and the law of variation of heat with time is obtained by deriving the previous function respect to time. The expression and the curve are given bellow:

$$q_v(t) = \frac{\left(\frac{0.0653 \cdot 4186.8 \cdot 250}{86400}\right)}{\left(0.0653 + 0.0175 \cdot \frac{t}{86400}\right)^2} \quad (8)$$

where t is expressed in days and q_v in W/m^3.

Finally, thermal boundary conditions have to be continuously updated during the construction process.

While the foundation is considered an adiabatic surface during the whole process, the external contour or the dam, at each construction stage, is modelled as a convective surface. In FLAC 3D, such boundary condition has the following form:

$$q_n = h(T - Te) \quad (9)$$

where q_n is the component of the flux normal to the boundary in the direction of the exterior normal, h is the convective heat-transfer coefficient [W/m^2 °C], T is the temperature of the boundary surface, and Te is the temperature of the surrounding fluid [°C].

4 RESULTS

4.1 *Preliminar checking*

As preliminary checking, for the working team to become more confident with FLAC 3D, two simplified mechanical models (without considering the construction sequence) where compared. Next Figures represent the maximum stresses (the less negative, thus meaning the highest values of tensile stresses) pattern using SAP2000 and FLAC 3D respectively.

The next Table shows the comparison of the obtained results in terms of maximum, minimum and vertical stresses.

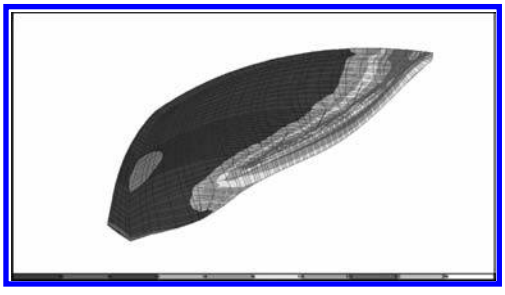

Figure 9. Pattern of maximum stresses with SAP 2000 NL.

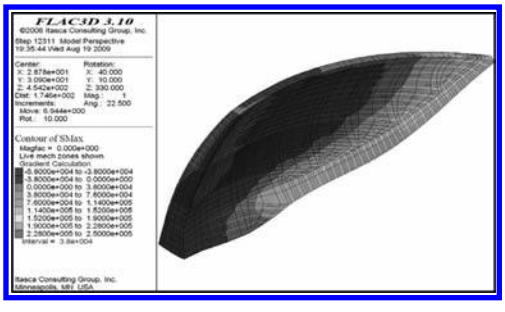

Figure 10. Pattern of maximum stresses FLAC 3D.

Table 3. Comparison between mechanical models from the two programs used in MPa.

	Diference (%)	
NODE	S min	S zz
57567	–8%	–9%
58135	–1%	–2%
57659	19%	19%
50010	–7%	–29%
50225	0%	–1%
50174	10%	10%
55028	30%	–12%
55149	9%	4%
55148	–21%	15%
1993	–27%	–25%
2400	2%	2%
2123	23%	16%
38	9%	10%
445	3%	2%
168	2%	0%

4.2 *Temperature and stress-history during construction*

The results for the complete model performed with FLAC 3D are shown in the following figures, where "control" joints have been grouped and ordered as follows (see next Figure) in order to present many of the graphics.

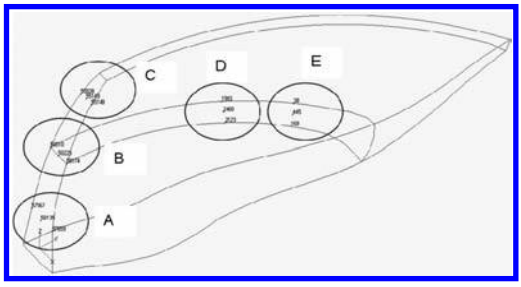

Figure 11. Scheme for graphic presentation of the results.

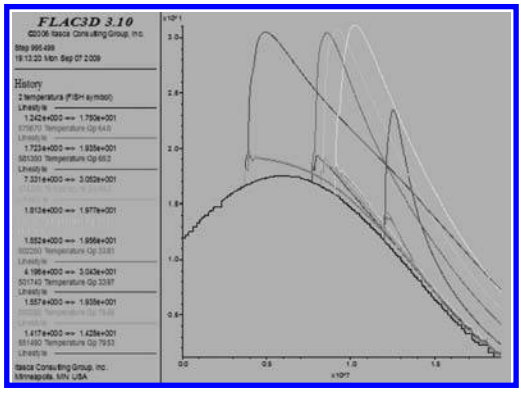

Figure 12. External and internal temperature (at the joints given by the formulators) during construction. X-axe is given in seconds. Y-axe is given in celsius degrees (×10).

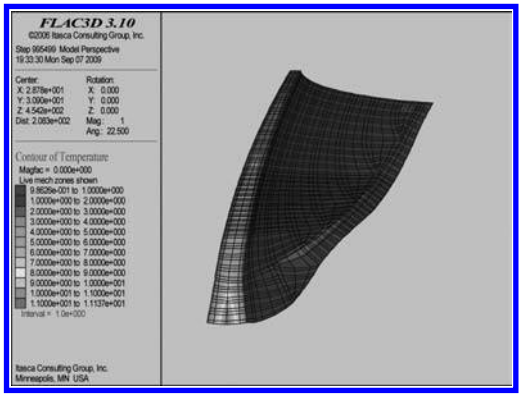

Figure 13. Final (at the end of construction) contour plot of temperatures (I).

Where A: 57567, 58135 and 57659 (plots in the same order), B: 50010, 50225 and 50174 (plots in the same order), C: 55028, 55149 and 55148 (plots in the same order), D: 1993, 2400 and 2123 (plots in the same order) and E: 38, 445 and 168 (plots in the same order).

Results are presented in this order in the following Figures:

- Temperatures
- Maximum (higher tensile) stress (final and joint histories)
- Minimum (higher compression) stress (final and joint histories)

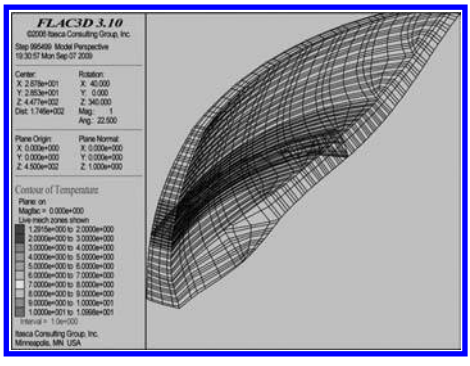

Figure 14. Final (at the end of construction) contour plot of temperatures (II).

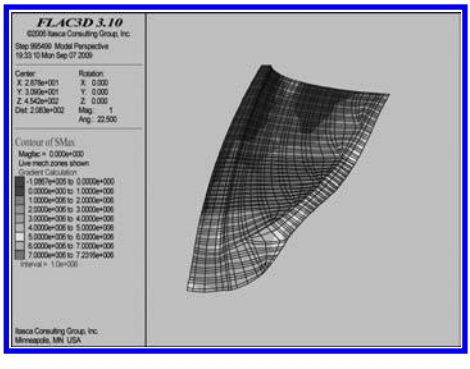

Figure 15. Final (at the end of construction) contour of maximum principal stresses (higher tensile) in Pa.

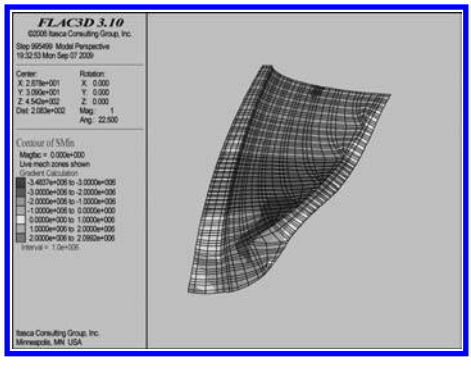

Figure 16. Final (at the end of construction) contour of minimum principal stresses (higher compressive) in Pa.

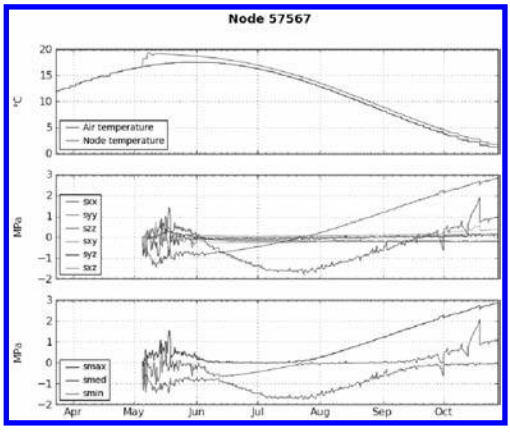

Figure 17. Temperature, stress tensor and principal stresses for node 57567 (zone A upstream).

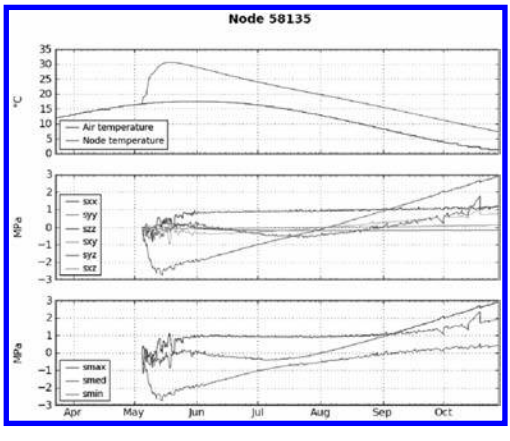

Figure 18. Temperature, stress tensor and principal stresses for node 58135 (zone A center).

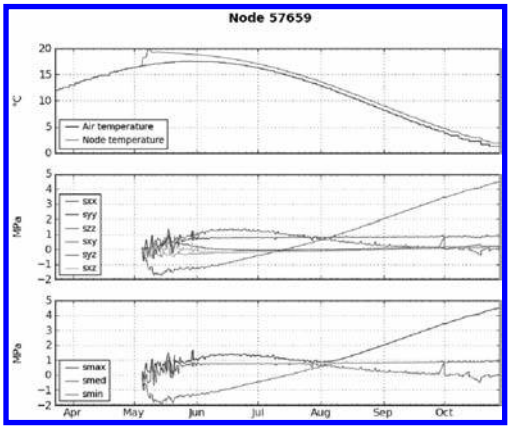

Figure 19. Temperature, stress tensor and principal stresses for node 57659 (zone A downstream).

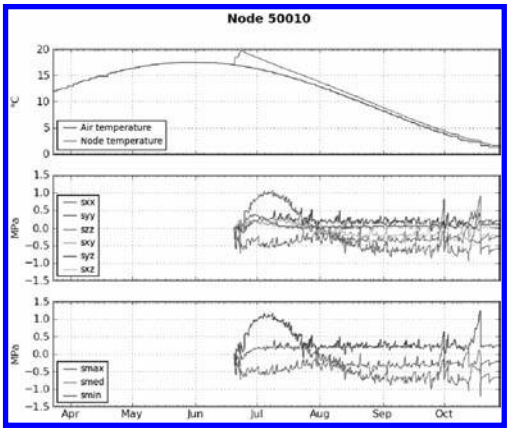

Figure 20. Temperature, stress tensor and principal stresses for node 50010 (zone B upstream).

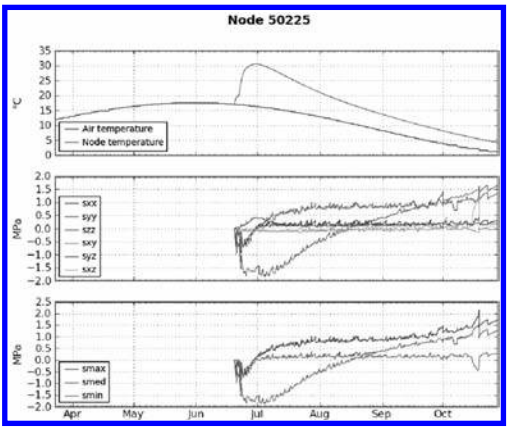

Figure 21. Temperature, stress tensor and principal stresses for node 50225 (zone B center).

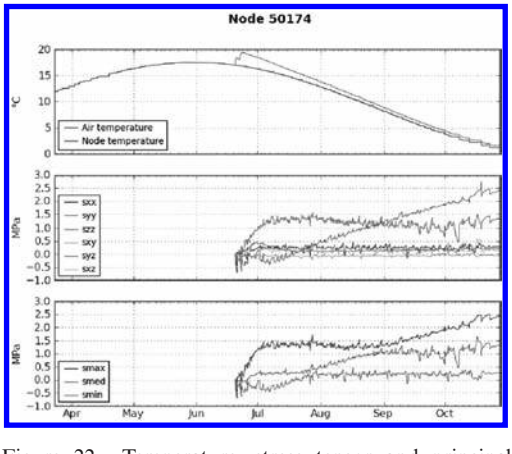

Figure 22. Temperature, stress tensor and principal stresses for node 50174 (zone B downstream).

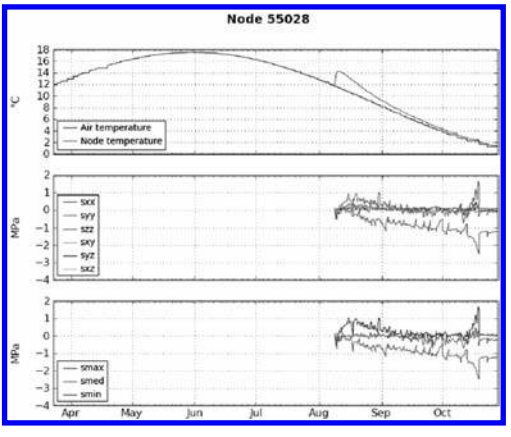

Figure 23. Temperature, stress tensor and principal stresses for node 55028 (zone C upstream).

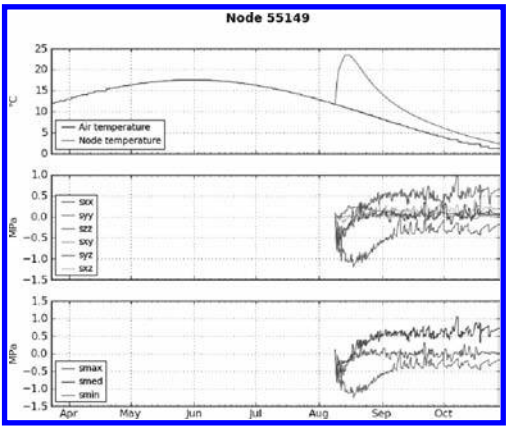

Figure 24. Temperature, stress tensor and principal stresses for node 55149 (zone C center).

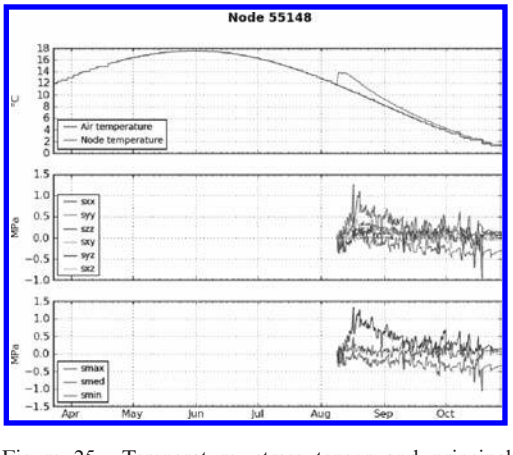

Figure 25. Temperature, stress tensor and principal stresses for node 55148 (zone C downstream).

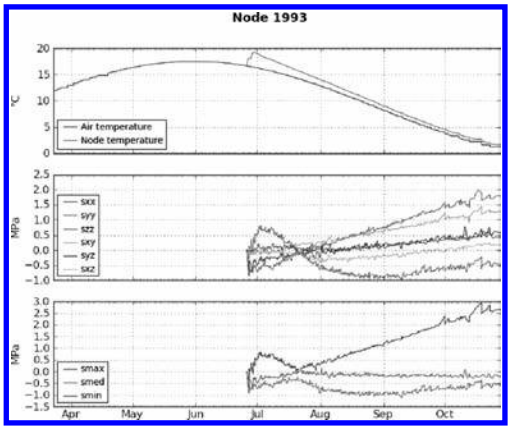

Figure 26. Temperature, stress tensor and principal stresses for node 1993 (zone D upstream).

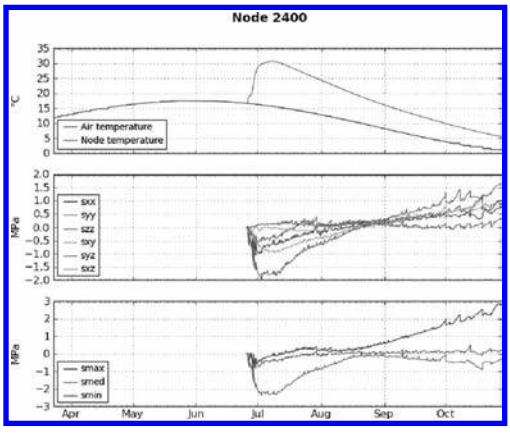

Figure 27. Temperature, stress tensor and principal stresses for node 2400 (zone D center).

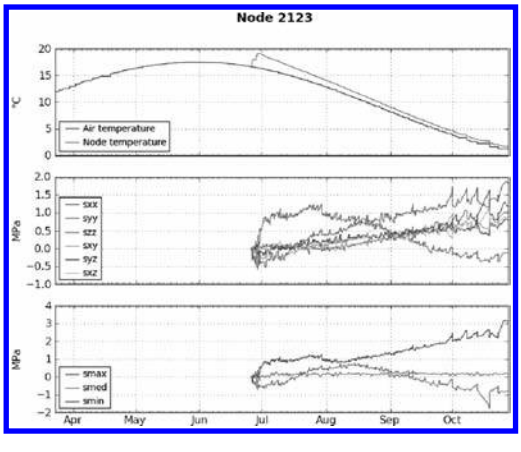

Figure 28. Temperature, stress tensor and principal stresses for node 2123 (zone D downstream).

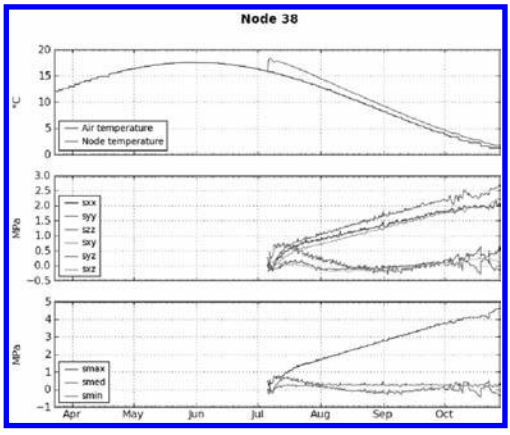

Figure 29. Temperature, stress tensor and principal stresses for node 38 (zone E upstream).

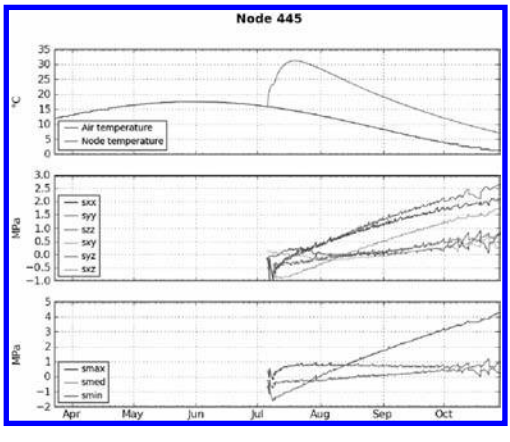

Figure 30. Temperature, stress tensor and principal stresses for node 445 (zone E center).

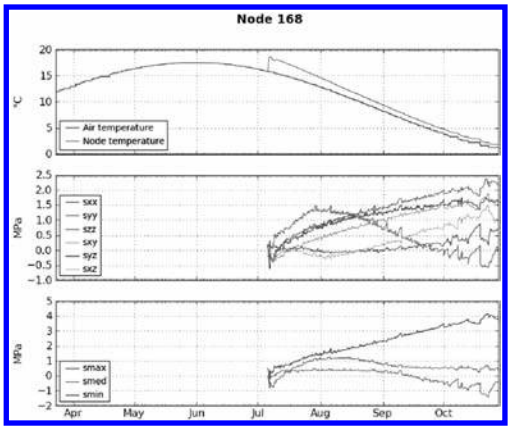

Figure 31. Temperature, stress tensor and principal stresses for node 168 (zone E downstream).

4.3 *Crack pattern*

Following Figures show the evolution on time of temperature, stress tensor, principal stresses and orientation of the maximum principal stress directions at two joints which have been considered as "representative" for each crack families:

– 50174 for the "horizontal" family.
– 168 for the "diagonal" family.

The two areas identified with a black circle correspond with the orientations of the maximum principal stress that can explain the cracks patterns observed. However, the tensile value reached is lower than the theoretical value of concrete's tensile rupture.

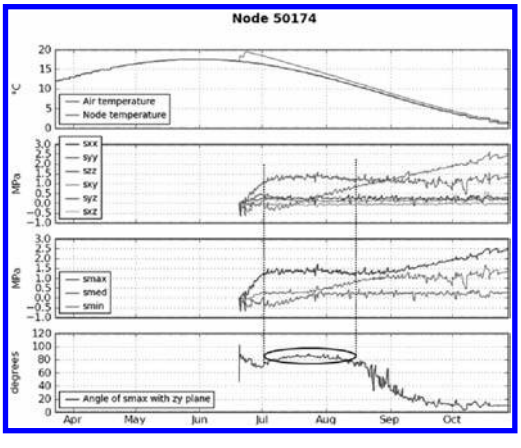

Figure 32. Evolution of temperature, stress tensor, principal stress and orientation of the maximum principal stress direction for node 50174.

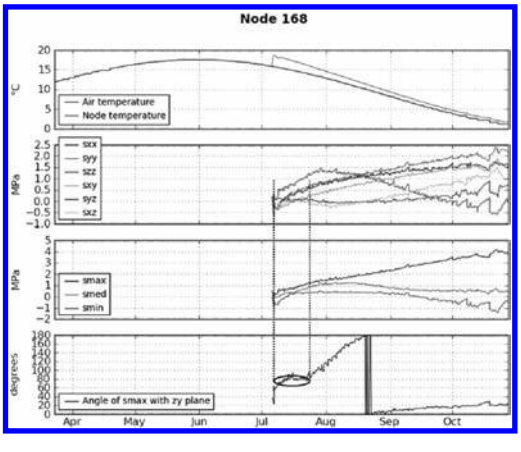

Figure 33. Evolution of temperature, stress tensor, principal stress and orientation of the maximum principal stress direction for node 168.

5 CONCLUSIONS

In order to study the thermal and mechanical phenomenon during dam's construction, an incremental linear elastic analysis has been realized. This means that a linear analysis is performed for each time step and when the equilibrium is reached the calculus moves to next instant of time.

FLAC 3D allows to perform a coupled analysis, i.e. at the same time it is examined the thermal and mechanical model. Therefore, for each calculation step temperatures and stresses should be balanced.

The performed model has been considered as a monolithic structure without joints. Using this model, the temperature variations of the dam and the stresses development due to the setting and hardening of concrete in real time can be simulated.

In the defined points, the temperature values obtained are good because the thermal behaviour expected can be explained by the analyzed model. Moreover, the value of stresses may be considered satisfactory due to the tensile zones appear where cracks have been observed. However, there are greater difficulties in defining the cracks direction, as the angular values which can be interesting are obtained with a tensile that does not produce a theoretically breaking.

This dam has been constructed in a short period of time—7 months—and therefore the cooling process of a concrete layer is not properly done before placing the next one. This can probably be a key factor for the internal generation of high stresses and the appearance of cracks.

Nobody really knows when cracks appeared, but the thermal model is able to point the tensile areas where there are cracks. The thermal gradient between the walls and inside the body can explain the stress generation in the dam.

Moreover, the dam's position with respect the sun and wind is not known. The radiation phenomenon could explain the cracks appearance if the sun rays strike on downstream face. Instead, the wind could improve the heat exchange between dam's walls and the environment.

Finally, the crack pattern obtained in this dam is parallel to the compressive forces generated and thus there is no need of being worried about. Furthermore, these cracks are similar to those seen when interstitial pressures are affecting the dam.

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Measured data generation for concrete dams by artificial neural network

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ABSTRACT: In this paper an Artificial Neural Network (ANN), a general back error propagating perception, is used to generate measured data for concrete dams. Since all of the measurements during the operation life of concrete dams have been read, it is possible to categorize these measurements due to various characteristics. These characteristics including: water level, temperature, and forces caused by water pressure, uplifting and loading. By using the data and having the corresponding conditions of dams a new data can be estimated. Through a parametric study, different geometry of dam shapes, water pressure, deformation and deflection the best network architecture for each case is obtained. The field data of pendulums and thermometers of a large dam in Iran for several years was employed to construct the neural networks. The results compared with the target field data and showed a good accuracy and adaptation with them.

1 INTRODUCTION

The artificial neural networks as a non-parametric system identification approach present a robust and efficient way to simulate the nonlinear behavior of engineering systems.

Artificial neural networks are collections of mathematical models that emulate some of the observed properties of biological nervous systems and draw on the analogies of adaptive biological learning. The key element of the ANN paradigm is the novel structure of the information processing system. It is composed of a large number of highly interconnected processing elements that are analogous to neurons and are tied together with weighted connections that are analogous to synapses.

Instrumentations are generally incorporated into dams for several reasons such as to follow the behavior of the dam during its construction, to compare the performance of the dam under operating conditions with that predicted by the designers and also to evaluate the overall condition and safety of the dam on a regular basis. They can also be used as a base experience to be used in the future projects. The relatively high cost of installation and conservation of these systems force the designers to provide the minimum required number of instrumentations. However minimum requirements exist due to the redundancy especially for embedded instruments because of the cost of repair or retrofit. Artificial neural networks as general tools can help the designer to increase the redundancy and also to cover the total behavior of dams.

2 BACK PROPAGATION NEURAL NETWORK

2.1 *Back propagation model*

There are different types of ANN. Back propagation is one of the most widely used of the neural network paradigms and has been applied successfully in applied studies in a wide range of areas. The back error propagating neural network is employed as the nonlinear identification approach in this study. These networks are introduced with three basic elements, which are nodes or units, layers or architecture and the activation function. The general configuration of these networks is shown in Figure 1. The bottom layer of units is the input layer, the only units in the network that

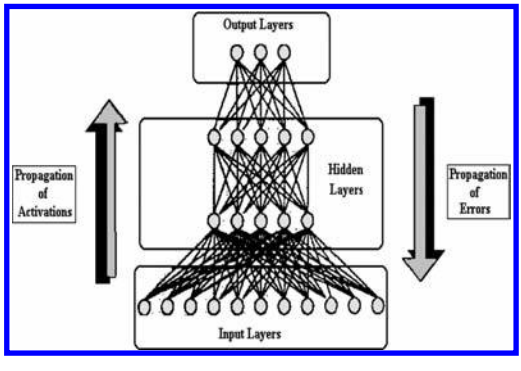

Figure 1. Back propagation network model (Dayhoff, 1990).

receive external input. The layer above is the hidden layer, in which the processing units are interconnected to layers above and below. The top layer is the output layer. In this network data are primarily entered through input layers which lead to output layers at this stage no back propagation is done. The output is compared with available empirical and analytical data, the error resulted from above is computed and finally distributed back wards. Therefore network relation function is reformed. In this circumstance, error propagation is done. The entire procedure is called network training. During training the weights associated with each interconnection are adjusted. After learning is completed the weights are fixed. A back propagation network is trained by supervised learning. The network is presented with pairs of patterns an input pattern paired with a target output. Upon each presentation, weights are adjusted to decrease the difference between the network's output and the target output. A training set, a set of input/target pattern pairs is used for training and is presented to the network many times. After training is stopped the performance of the network is tested. The back propagation learning algorithm involves a forward propagating step followed by a backward propagating step. Both the forward and back propagation steps are done for each pattern presentation during training. ANN produces precise response in limited training however beyond the so-called limits may not give as accurate responses. Hence they have to be train various patterns. One of the problems with ANN is the number of training (Dayhoff, 1990).

2.2 *Back propagation algorithm*

The procedure of back propagation neural networks is briefly explained here (Bishop, 1995). Set all weights, biases, weight modifiers and bias modifiers to random values in the desired ranges then scale and present input vector to the input layer. Calculate input vector of the hidden layers:

$$Y_u^{in} = \nu_u + \sum_{h=1}^{H} x_h \cdot w_{hu} \tag{1}$$

where Y = The input or output of the Hth hidden; Kth output layers; ν = Biases; $X = (x_1, x_2, \ldots x_i)$ is the input vector with i members; and W is weight.

Determine output vector using the transferfunction:

$$Y_u^{out} = f(Y_u^{in}) \tag{2}$$

Continue the procedure for all layers to obtain output vector then calculate the error vector and total error to check for convergence:

$$\delta_u = (t_u - y_u) \cdot f'(Y_u^{in}) \tag{3}$$

where δ is error in the layers; $T = (t_1, t_2, \ldots, t_U)$ is the target vector with u members:

$$E = \frac{1}{2}\sum_{u=1}^{U}(t_u - y_u)^2 \tag{4}$$

Calculate weight and bias modifiers:

$$\Delta w_{hu} = \alpha \cdot \delta_u \cdot y_h \tag{5}$$

$$\Delta v_{hu} = \alpha \cdot \delta_u \tag{6}$$

where E = Total error; and α is learning rate.

Modify weights and biases in the output layer:

$$w_{hu}^{new} = w_{hu}^{old} + \Delta w_{hu} \tag{7}$$

$$\nu_{hu}^{new} = \nu_{hu}^{old} + \Delta \nu_{hu} \tag{8}$$

Back propagate error in the hidden layers of the network, modify weights, biases, weight modifiers and bias modifiers and go to input vector of the next hidden layers.

2.3 *Network training methodology*

For successful training, weighted relation of network should not be close to unites. An appropriate solution is to use a difference between goal activity and computed activity for weighting adjustment and error reduction. This is called "rule of Delta". In Delta Rule the weight varies between i and j provided the output is not zero or variation between output and real values. There are two methods for Delta Rule: The Least Mean Square (LMS), the threshold method.

The least mean square is applied when output units are non-linear. The total error is the sum of squares (Masri, et al. 2000):

$$E = \sum_p Ep = \sum_p \sum_i (tpi - upi) \tag{9}$$

where E = error; p = Training pattern; t_{pi} = Target value; and u_{pi} = Real output.

In a back propagation neural network the error resulted from training and weights correction is called a loop. Network obtains data from input and determines output units weight for training. Output errors are computed and propagated back into the network. At this stage, modify input related weights by analysis the errors. While a number of loops make up an epoch, network training may consists of hundred or even thousands of epoch. The Delta Rule minimizes sum of LMS by gradient reduction. The results obtained by this

method depend on the errors values. If the error values are too small, the network converges and hence result is obtained conversely. Therefore, at first, ANN architecture is selected then training patterns is determined and it is trained by Delta Rule. Finally, the network is tested which means that how network responds to new input (Flood & Kartam, 1994).

3 CASE STUDY

The case study considered here is a double curvature concrete arch dam in Iran. Jiroft dam has been located near the Jiroft city in southern Iran. The construction of this concrete dam was finished in 1988. The dam has 133 meters height and 250 meters crest length as shown in figure 2. The dam has now two pendulums figure 3. figure 4 shows the instrument locations in the dam and in figure 5 a sample recorded data for one year with its design limits have been presented (Jiroft dam report 2003).

Due to the space consideration, some of the results are selected. In order to perform the adaptive ascending algorithm, were applied to the program and the number of hidden layers and hidden layer neurons were changed. As shown in figure 6 for five hidden neurons the networks with one, two and four hidden layers were converged. After each convergence the effect of over training or over fitting may be seen. In the next step for the selected five hidden layers different numbers of hidden neurons were examined which the results of some of them are presented in figure 7. The network with five, seven and ten hidden neurons was converged and the first one was selected. So the final network architecture shown in figure 8

Figure 2. Jiroft concrete arch dam in south of Iran.

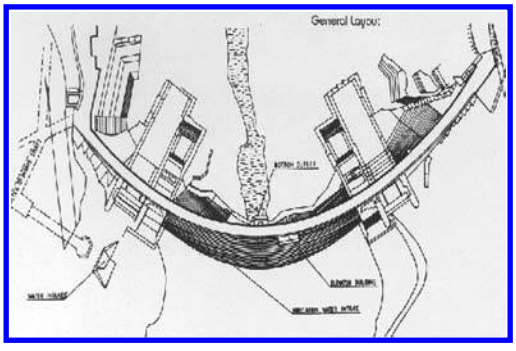

Figure 3. Site plan of Jiroft dam.

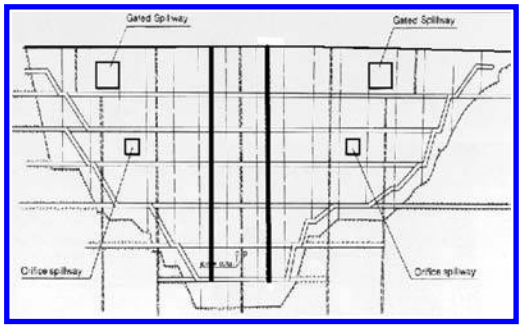

Figure 4. Pendulum locations in the dam section.

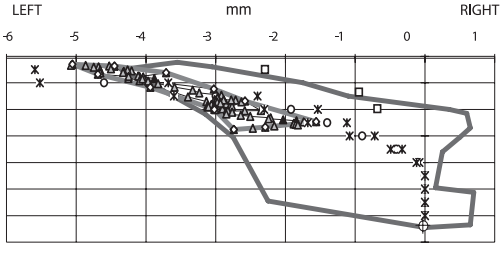

Figure 5. Sample pendulum recorded data compared to the allowable values (Jiroft dam report 2003).

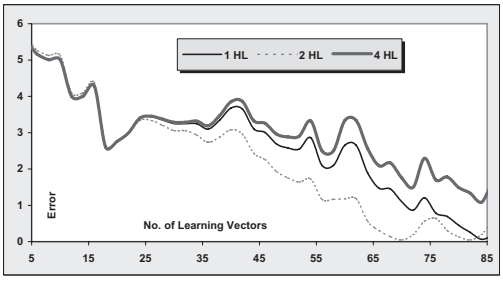

Figure 6. Network error for different number of hidden layers.

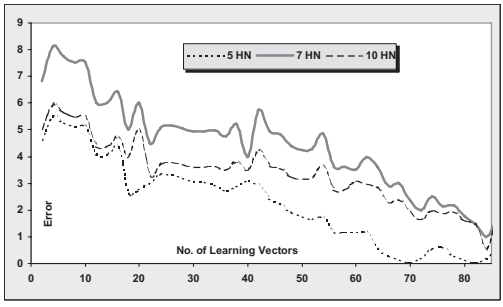

Figure 7. Network error for different number of hidden neurons.

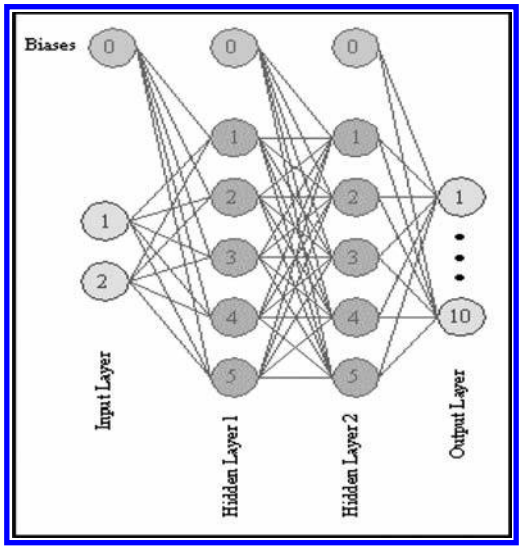

Figure 8. Final network architecture.

contained two hidden layers and five hidden neurons in each layer.

In the dam monitoring additional focus should be placed on those instruments that do not appear functioning properly and further investigating regured. For example, data should be checked against redundant data to determine whether trends and magnitudes are the same or not. Calibration of the instruments should be checked. Often, tests can be devised to evaluate proper functioning. The network can be used for this problem in the following way.

For the test phase, if there is a discrepancy between the measured behavior of the dam and those estimated by the network, it may indicate following problems, the dam is not performing satisfactorily, failure is developing or occurring, the data or observations do not adequately represent the behavior of the dam, and exiting conditions were not accounted for the expected behavior. In each cases it is conventionally useful to perform field investigations and install additional instrumentation to evaluate the behavior. Note again that rational judgment must be used to take action rather than do further investigation. If what is going on is serious enough you could put in more instruments just to see the dam fail. But the neural network can directly tell us whether such problems may have been occurred.

4 CONCLUSIONS

A general back error propagating perception was used to model the structural deformations of a concrete dam. The recorded field data was implemented to create the data for supervised learning procedure. The input units were water level and week number in the year and the outputs of the network were the displacement response of ten monitored points. The adaptive ascending algorithm with full connections and bias terms were used to construct the best network architecture. From the results presented in this study it is concluded that neural networks show great promise in becoming useful tools in practical structural analysis applications.

Generally, if there is a discrepancy between the measured behavior of the dam and those estimated by the neural network, conventionally it is useful to perform field investigations and install additional instrumentation to evaluate the behavior while the neural network can directly predict whether such problems may have been occurred. On the other using the extrapolation ability of the artificial neural network it can predict the future expected response of the structure in an arbitrary time and an inexperienced water level. It is recommended that for full coverage of the dam behavior all the instrumentations be considered in the construction of neural network. Finally the artificial intelligence can help us with the monitoring.

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Analysis of a gravity dam considering the application of passive reinforcement

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ABSTRACT: Small concrete gravity dams are sometimes built using passive steel anchors to enhance their stability for the applied loads. It is important to assess the amount of internal tensile and shear forces resisted by the anchors to verify their structural adequacy. This paper presents the application of the distinct element method to study the magnitude and distribution of internal anchor forces. Comprehensive parametric analyses considering the anchor shear stiffness and the friction angle of the dam-foundation interface have been performed on a small dam 3 m high. The case of several anchors along the section was also investigated. It is shown that (a) the required displacements to induce anchor loads are very small (of the order of 10×10^{-3} mm of crack opening), (b) that the anchor shear stiffness has small impact in the load share between anchor and concrete, and finally (c) that strength properties of rock-concrete interface play a key role in the structure behavior.

1 INTRODUCTION

Several existing small concrete gravity and buttress dams (less than approximately 7 m high) have been constructed with passive steel reinforcement anchors to ensure adequate structural stability for usual, unusual, and extreme load combinations (Fig. 1).

It is also possible to consider adding passive reinforcement to existing small gravity dams that do not meet current stability criteria. The basic advantages of using passive reinforcement to a section subjected to axial (P) and shear loads (H) as well as bending moments (M) are (a) the increase in concrete compressive force resultant thus increasing the shear strength that can be mobilized, V_c, (b) a small decrease in crack length, $L_{cr,}$ as compared to unreinforced sections, and (c) the direct dowel action of the anchor, V_a, increasing the shear strength of the section (Fig. 2).

However, the use of passive anchors presents also some disadvantages. A first objection is that a small amount of displacement is required to induce axial and shear forces in the anchors (USACE 2005). Durability, that is mainly deterioration due to corrosion, is of concern, fatigue has also been discussed however, the induced stress level are low as well as the stress variations due to water level fluctuation, seasonal temperature variations and ice loads for dams located in northern regions.

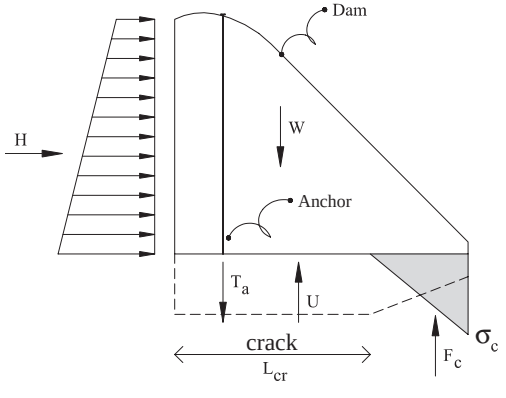

Figure 1. Gravity dam with passive anchor (H—Hydrostatic load, W—Weight, T_a—Tensile force, U—Uplift force, σ_c—Concrete stress, F_c—Concrete force, L_{cr}—Crack length).

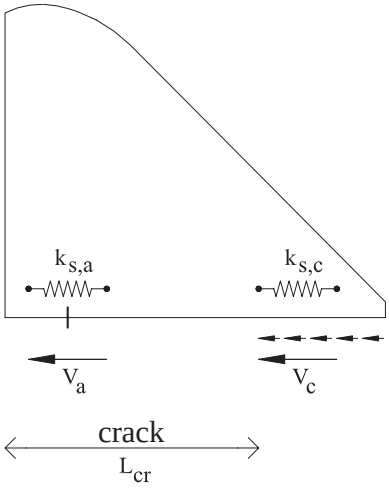

Figure 2. Shear forces and stiffnesses (H—Hydrostatic load, V_a—Anchor shear force, V_c—Concrete shear force, $k_{s,a}$—Anchor shear stiffness, $k_{c,a}$—Concrete shear stiffness, L_{cr}—Crack length).

To assess the integrity of the anchor one needs to compute the tensile, T_a, and shear force, $V_{a,}$ that will be induced by the applied loads including uplift pressures. Interaction formulas can then be applied:

$$\left(\frac{T_a}{T_{a,\max}}\right)^2+\left(\frac{V_a}{V_{a,\max}}\right)^2\le 1 \tag{1}$$

The purpose of this paper is to present a computational procedure using the distinct element method (a) to assess the tensile force developed in the anchors, T_a, and how the applied total shear force H is split among the anchor, V_a, and the concrete, V_c, contributions. These forces represent the driving shear forces. Secondary failure mechanisms should be verified to ensure the integrity of the anchor (pull-out of a rock mass thus uplifting of the anchor, bond slip at the various interfaces, bearing failure (Wyllie 1999)), however, this is outside of scope of this paper. A sliding safety factor could be determined from the ratio of the maximum shear force that could be mobilized to the applied shear load. The paper is organized as follows. We first present formulations to compute the axial and shear stiffnesses as well as the maximum strength that could be developed in the anchors. The distinct element method, used to solve the equilibrium equation with a concrete-rock interface reinforced by passive anchors, is outlined.

An extensive parametric analysis is performed to assess the effects of the shear stiffness of the anchor and the friction angle of the concrete-rock interface on the distribution of the shear load at the rock-concrete interface. It is shown that (a) the required displacements to induce anchor loads are very small (of the order of 10×10^{-3} mm of crack opening), (b) that the anchor shear stiffness has small impact in the load share between anchor and concrete, and finally that strength properties of rock-concrete interface play a key role in the structure behavior.

2 MECHANICAL AND STRENGTH PROPERTIES OF PASSIVE ANCHORS

For gravity dams, to assess a sliding failure scenario on a discontinuity interface it is interesting to identify the contribution of passive anchors in the total shear strength. The action of a passive anchor, fully grouted, increases the compressive force developed in the interface. Dowel action and increased frictional forces are developed when a small relative displacement between the two sides of the discontinuity under consideration is taking place (Pells 1974, Haas 1976, Azuar et al. 1979).

To develop a model for anchor behavior, the shear stiffness of the passive anchor must to be determined. There are several possible approaches as, for example, one proposed by St. John and Van Dillen (1983), who considered the bar as a short beam,

$$K_{s,a}=\frac{3E_bI}{2L_d^3} \tag{2}$$

where,

E_b—Young's modulus of the bar
I—moment of inertia of the bar
L_d—decay length of the bending stress in the bar

To take account with the grout effect, Lorig (1985) proposed the following equation

$$K_{s,a}=E_bI\,\beta^3 \tag{3}$$

where,

$$\beta=\left(\frac{K}{4E_bI}\right)^{\frac{1}{4}} \tag{4}$$

$$K=\frac{2E_g}{\left(\frac{d_1}{d_2}-1\right)} \tag{5}$$

I—moment of inertia of the bar
E_g—Young's modulus of the grout
d_1—reinforcement diameter
d_2—hole diameter

The axial stiffness of the anchor could be first estimated as AE/L_d more complicated formulas have been presented in UDEC User's Manual (2004) and involves the grout shear modulus as well as the Young's modulus of the steel reinforcement

Based on the tests carried by Bjurstrom (1974), an empirical rule to estimate the maximum anchor shear resistance was developed,

$$V_{ult} = 0.67 d_1^2 \sqrt{\sigma_b \sigma_c} \tag{6}$$

where,

σ_b—uniaxial compressive strength of rock
σ_c—yield strength of bar

To provide an assessment of the safety condition of the dam in terms of shear failure, a global Safety Factor (SF) could be estimated, which compares the shear resistance and driving shear force, given by

$$SF = \frac{(V_c + V_a)}{H_{us} - H_{ds}} \tag{7}$$

where,

V_c—Concrete shear force
V_a—Anchor shear force
H_{us}—Upstream hydrostatic force
H_{ds}—Downstream hydrostatic force

To apply this formulation one needs to know V_a and, by corollary, V_c. Theoretically, for small (fairly rigid) gravity dams with passive anchors, the structure may be assumed as two springs in parallel, one represents the anchor shear stiffness $K_{s,a}$ and the other the concrete shear stiffness $K_{s,c}$. Thus V_a and V_c are determined by

$$V_a = \frac{H(K_{s,a})}{K_{s,a} + K_{s,c}} \tag{8}$$

$$V_c = \frac{H(K_{s,c})}{K_{s,a} + K_{s,c}} \tag{9}$$

where,

$$H = H_{us} - H_{ds} \tag{10}$$

Because the anchor behavior depends on the passive anchor slip in the plane of the discontinuity and the relative stiffness of the anchor and concrete it is of interest to address this problem using numerical tools that allow the explicit modeling of axial and shear displacement, such as those based on distinct element method.

3 DISTINCT ELEMENT METHOD FOR GRAVITY DAM ANALYSIS

The Distinct Element Method (DEM) was initially developed for analysis of geotechnical problems, particularly in the solution of models that have high discontinuity, with importance in the process of rupture. DEM allows explicit modeling of joints and use of appropriate constitutive models to them. This feature is mainly reflected in the ability of DEM to predict the mechanisms and loads displacement response up to failure.

The solution is based on time integration of the equation of motion (2nd Newton's Law) of each degree of freedom, using the method of central difference. Static and dynamic analyses are performed using the same calculation scheme. The static response is obtained by a dynamic relaxation process by means of applying a damping force sufficiently high to dissipate kinetic energy and reach the solution of static equilibrium.

The numerical application developed in this work, designated as DEC-DAM (Distinct Element Code for gravity DAM analysis), is an implementation of DEM to gravity dam analyses. Using deformable blocks, discretized by quadrilateral finite elements (four sides and four nodal points), with shape functions of 1st order, whose integration is done with four Gauss points. The contact between the blocks is of the type face-to-face and adopts a linear distribution of stress.

The constitutive model of joints is following a Mohr-Coulomb criterion, defined from the parameters of cohesion, friction angle, tensile strength and normal and shear stiffnesses that control elastic deformation of the joints. In general, the residual value of cohesion is zero, after exceeding the maximum value of shear stress (peak value), the shear strength becomes conditioned only by the friction angle. The concrete normal stress is limited by its tensile strength very often taken as zero at the dam-foundation interface. After joint opening, the contact could still remain active, and is able to model shearkeys or another interlock mechanism.

One of the features specific to the case of gravity dams is the need to include uplift pressure on a cracked surface. The interfaces under study are parameterized and is possible throughout the analysis obtain specific information, for example the crack length, beside if the crack still in progress, the uplift pressure can then be updated.

Another tool is the modeling of passive anchors between two blocks (local reinforcement). After installing an anchor, the applied forces (normal and shear) in the blocks, depends on their relative displacement. The behavior of anchor is controlled by the axial and shear stiffnesses, axial and shear ultimate capacities and axial and shear failure strains.

Throughout the development of DEC-DAM, validation tests are carried out with 3DEC (2003) and UDEC numerical applications, both developed by Itasca. Similar procedure was followed in this study, whose results were confirmed by 3DEC (Bretas et al. 2010a and 2010b).

4 APPLICATION TO A SMALL EXISTING DAM

4.1 *System analyzed*

The structure under study is a small dam (Fig. 3) located in Canada, with an overflow profile, height 2.9 m and base width of 3.65 m, laid out in a wide valley in a straight line plan position, without vertical contraction joints or drainage system. The section is reinforced with a passive anchor (Fig. 4), 29.9 mm diameter, positioned approximately 0.6 m from the upstream heel.

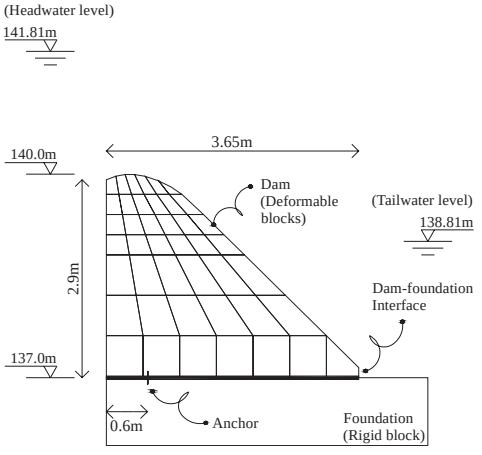

Figure 3. DEC-DAM model with geometry definition and load case.

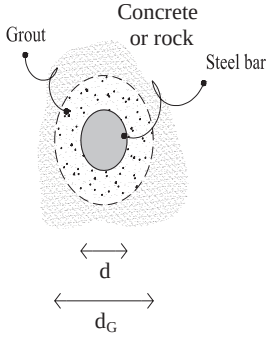

Figure 4. Anchor cross-section (30 M Steel Bar) (d = 29.9 mm, σ_y = 400 MPa, A = 700 mm^2, E = 200 GPa, G = 18 GPa, d_G = 64 mm).

Analysis was made for a flood scenario, with headwater level of 141.81 m (height 4.81 m) and tailwater level of 138.81 m (height 1.81 m). On the upstream face, hydrostatic pressure and uplift were considered as usual. However, regarding the downstream face, only the effect of uplift pressure was considered, because in this case, hydrostatic pressure works as stabilizing load and was disregarded.

Materials properties are described in Table 1. The foundation block, where dam-foundation joints are resting, was considered rigid. To analyze the effect of sharing shear loads between foundation and anchor, we adopted different values for the anchor shear stiffness (50, 350, 550, 1000 and 2000 MN/m). These values reflect a range of parameters that could be inserted in St. John and Van Dillen (1983) and Lorig (1985) equations. For the friction angle of the rock-concrete contact interface we considered a wide range of possible values (30°, 37.5°, 45°, 52.5° and 60°).

During the analysis an uplift update scheme was adopted in cracked zone along the base of the dam. In all cases, the final crack length corresponded to about 1 meter (approximately 29% of the total width), equivalent to an uplift pressure increase of about 16 kN, or approximately 13% of the initial uplift pressure. Total applied loads are shown in Table 2.

4.2 *Effects of shear stiffness of the anchor*

Parametric analyses were made of the anchor shear stiffness for the values of 50, 350, 550, 1000

Table 1. Properties of the materials.

Parameter	
Concrete volumetric weight	24 kN/m^3
Water volumetric weight	10 kN/m^3
Rock-concrete (R-C) cohesion	null
R-C friction angle	various
R-C normal stiffness	10 GPa/m
R-C shear stiffness	3.33 GPa/m
Concrete Young's modulus	20 GPa
Concrete Poisson's ratio	0.2
Anchor Young's modulus	200 GPa
Anchor Poisson's ratio	0.29
Anchor axial stiffness	3000 MN/m
Anchor shear stiffness	various
Anchor axial ultimate capacity (tension)	400 MPa
Anchor shear ultimate capacity	168 kN

Table 2. Total request loads.

Load description	[kN]
Dam self-weight	164.4
Downstream hydrostatic pressure	97.4
Uplift (initial condition)	120.8
Uplift (after crack)	136.7

and 2000 MN/m, keeping constant the properties of the foundation plan, with a friction angle of 45° and null cohesion.

The results are shown in Table 3 and displacements observed in anchor and foundation in Table 4. The values of the shear forces in the anchor and along the foundation are shown in Figure 5. The tension that developed in the anchor is shown in Figure 6, for the different shear stiffness values examined.

Table 3. Normal and shear forces in the anchor and in the concrete for different shear stiffnesses of the anchor (the concrete friction angle is constant, $\phi = 45°$) [kN].

$K_{s,a}$ [MN/m]	V_a	T_a	V_c	F_c
50	45.3	25.0	52.5	–52.7
350	44.8	25.0	53.0	–52.7
550	45.2	25.0	52.7	–52.7
1000	46.0	25.1	51.8	–52.8
2000	46.3	25.2	51.9	–52.9

Table 4. Normal and shear displacements in the anchor and in the concrete for different shear stiffnesses of the anchor (the concrete friction angle is constant, $\phi = 45°$) [$\times 10^{-6}$ m].

$K_{s,a}$ [MN/m]	dV_a	dT_a	dV_c	dF_c
50	910.0	8.3	910.0	–2.6
350	130.0	8.3	130.0	–2.2
550	82.1	8.3	83.3	–2.6
1000	45.9	8.4	47.3	–2.6
2000	23.1	8.4	24.6	–2.7

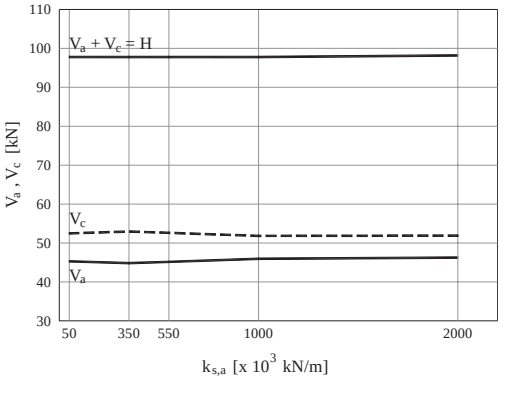

Figure 5. Shear forces in the anchor and in the concrete for different shear stiffnesses of the anchor (the concrete friction angle is constant, $\phi = 45°$).

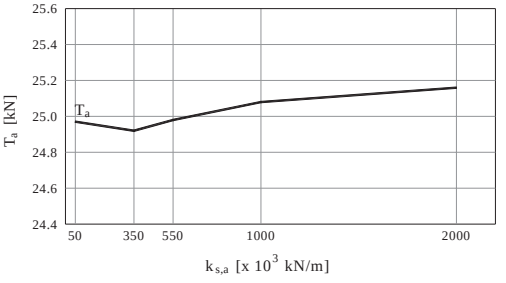

Figure 6. Tensile force in the anchor for different shear stiffness of the anchor (the concrete friction angle is constant, $\phi = 45°$).

Table 5. Normal and shear forces in the anchor and in the concrete for different concrete friction coefficients (angles) (the anchor shear stiffness is constant, $K_{s,a} = 1000 \times$ MN/m) [kN].

ϕ	V_a	T_a	V_c	F_c
30°	66.0	27.3	31.8	–55.0
37.5°	56.3	26.2	41.5	–54.0
45°	46.0	25.1	51.8	–52.8
52.5°	33.0	23.6	64.8	–51.4
60°	19.6	22.1	78.2	–49.9

Table 6. Normal and shear displacements in the anchor and in the concrete for different concrete friction coefficients (angles) (the anchor shear stiffness is constant, $K_{s,a} = 1000$ MN/m) [$\times 10^{-6}$ m].

ϕ	dV_a	dT_a	dV_c	dF_c
30°	66.0	9.1	71.1	–3.0
37.5°	56.3	8.7	59.6	–2.8
45°	45.9	8.4	47.3	–2.6
52.5°	32.9	7.9	31.8	–2.4
60°	19.6	7.4	15.9	–2.1

From these data it appears that the internal forces are not sensitive to anchor shear stiffness variation.

4.3 *Effect of friction angle at the rock-concrete interface*

Another parametric analysis was also made but now in relation to the friction angle (30°, 37.5°, 45°, 52.5° and 60°) of the rock-concrete interface for a null cohesion and constant anchor shear stiffness, equal to 1000 MN/m. The results are presented in Table 5 and displacements observed in Table 6.

Figure 7 represents the results of the shear forces in the anchor and in the foundation and Figure 7 the axial forces in the anchor.

In this case, that involves changes in the shear strength characteristics of the concrete-foundation

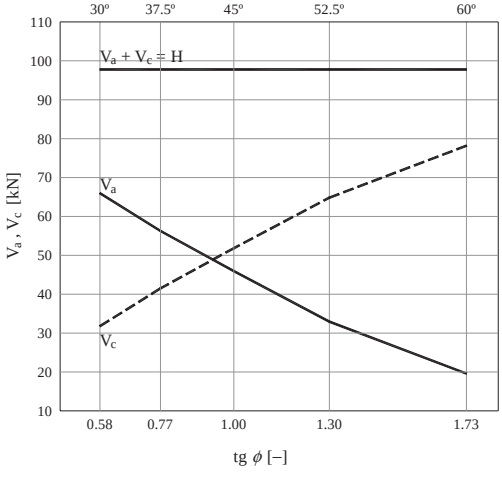

Figure 7. Shear forces in the anchor and in the concrete for different concrete friction coefficients (angles) (the anchor shear stiffness is constant, Ks = 1000 MN/m).

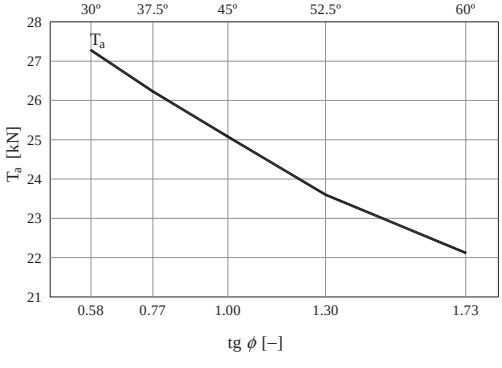

Figure 8. Tensile force in the anchor for different concrete friction coefficients (angles) (the anchor shear stiffness is constant, Ks = 1000 × 103kN/m).

interface, it is possible to observe an increase of shear load split to the foundation, discharging the anchor as the friction angle is increased.

4.4 *Effect of multiple anchors*

A second anchor was included to original model (Fig. 9). The friction angle adopted for the rock-concrete interface was 45°, while anchor shear stiffness was 1000 MN/m, for both anchors. The remaining properties, as well as loads, match those already described. The final crack is similar to that observed in previous analysis, i.e. about 29% of the base was cracked. The results are presented in Table 7.

The comparison (Table 8) between 'one-anchor' and 'two-anchors' models shows an increase in the compressive stress on the concrete in the 'two-

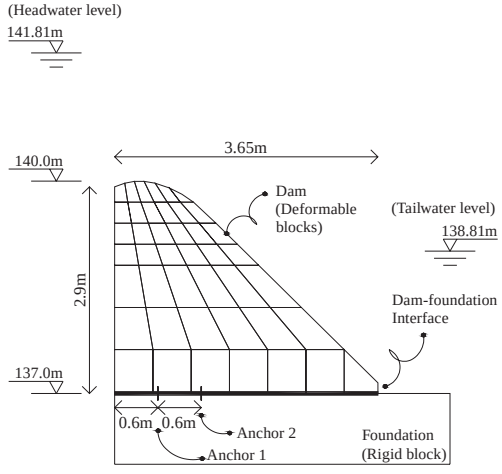

Figure 9. DEC-DAM 'two-anchors' model with geometry definition and load case.

Table 7. Normal and shear forces in the anchor and in the concrete for (the anchor shear stiffness and the concrete friction angle are $K_{s,a}$ = 1000 MN/m and ϕ = 45°, respectively).

Result	Force [kN]	Displ. [×10^{-6} m]
Axial force, Anchor 1 ($T_{a,1}$)	21.6	7.2
Shear force, Anchor 1 ($V_{a,1}$)	17.9	17.9
Axial force, Anchor 2 ($T_{a,2}$)	13.6	4.6
Shear force, Anchor 2 ($V_{a,2}$)	17.4	17.4
Axial force, Concrete (F_c)	–63.0	–2.75
Shear force, Concrete (V_c)	62.5	17.3

Table 8. Summary of results for models 'one-anchor' and 'two anchors' (the anchor shear stiffness and the concrete friction angle are $K_{s,a}$ = 1000 MN/m and ϕ = 45°, respectively).

#Anchors	$T_{a,1}$	$T_{a,2}$	F_c	$V_{a,1}$	$V_{a,2}$	V_c
1	25.1	–	–52.8	46.0	–	51.8
2	21.6	13.6	–63.0	17.9	17.4	62.5

anchors' case, therefore concrete shear force is also larger. Thus the anchor shear force ($V_{a,1}$ = 46.0 kN) for the case 'one-anchor', is greater than the sum of the anchor shear forces ($V_{a,1}$ + $V_{a,2}$ = 17.9 + 17.4 kN), for the case 'two-anchors'.

5 CONCLUSIONS

This paper presented the magnitude and distribution internal shear, tensile, compressive forces that

develop in small gravity dams with passive steel anchor reinforcement. The distinct element method was used to obtain a solution that is in equilibrium, respect the compatibility of deformations and follow the material laws. It was found that:

1. The shear stiffness of the anchor does not have a significant effect on the internal load magnitude and distribution.
2. The computed displacements to mobilize the internal forces are extremely small (of the order of 10^{-5} m). This is a significant result because it is clearly shown that "large" displacements are not needed to develop significant loads.
3. The dam-foundation friction angle is the most important parameter that will control the amount of shear load resisted by the anchor, while some variations in anchor tensile forces are noted.
4. In the case of multiple anchors the effective lever arm of the anchors is reduced thus increasing the concrete normal force and related shear force resisted by the concrete.

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Long term performance

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The adverse affect of expedient decision in design and construction on the long term performance and safety of embankment dams

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ABSTRACT: Embankment dams, once constructed and in service, tend to be viewed as inert structures that will last for thousands of years with minimal maintenance. Of course, the actual experience is different as reflected in statistics assembled by ICOLD that reveal about half of the failure incidents related to dams that have been in service for an extended period. In the case of earth dams, about two-thirds of these long-term failures developed through deterioration processes. This paper looks to the underlying origin of some deterioration processes through four case histories of engineered dams as revealed through their instrumented performance and required remedial works. The focus of the paper is the choices that lead to these earth dams being vulnerable to internal erosion and dam safety. In the cases considered, two factors appear contributing errors: i) adoption of novel design without adequate supporting engineering ("self filtering" is particularly implicated); and, ii) expedient design and construction choices to keep a project "on time, on budget". Both factors are associated with a seeming lack of appreciation for the extended timescales of earth dam service and experience. Based on these case histories, it is suggested that safe stewardships of earth dams consider more than the physical aspects of the dam (as documented in construction records and subsequent inspections during service) and in particular assess the context of the dam's design and construction. An expansion of the failure modes used as a basis of dam safety reviews is suggested to include aspects of the design and construction choices, leading to better identification of potential deficiencies at an early stage.

1 INTRODUCTION

Although the rate of failure and occurrence is modest, embankment dams do fail and the consequences can be so large that even small rates of failure present unacceptable risk to the public and community at large. This leads to the engineering consideration for public safety and the question '*why do earth dams fail*'? The causes of water-retaining earthfill dam failures have been investigated and summarized by the International Committee on Large Dams (ICOLD) based on the history of actual dam failures. This study (ICOLD, 1995) revealed that half the risk of dam failure arises during construction and first impoundment; once through that part of a dam's life there is an ongoing risk of failure over time—dams are not static, inert structures but age over time. In terms of failure causes, the study revealed that slightly more than a third of failures were caused by overtopping; of the other failure modes, all related to geotechnical aspects including internal erosion (in either embankment and/or foundation), seepage/piping (in embankment and/or foundation), and unexpected settlements (which may have caused rupture of a culvert/pipe through the dam). Given these mechanisms of dam failure, the question is posed "what is there about the engineering or construction of embankment dams that leads to these failure modes developing"? It cannot be said that dams are constructed without appropriate engineering since in general, from about the 1850s, most dams of any significant size have been engineered. What then could be going wrong?

Geotechnical engineering invariably requires design with uncertain and limited information, so that part of the dam failure risk can be attributed to random occurrence of unfortunate conditions. There can also be unappreciated or incorrectly assessed technical factors, and human errors that are discussed in Petroski (1994). In the context of earthdams, an example of incorrectly assessed technical aspects includes the concept of using 'self filtering' gradations in preference to layered filter-drain systems that has lead to dams being constructed that are in their operating future vulnerable to internal erosion. There can also be failure in recognizing untoward situations such as the

failure to appreciate the significance of seepage from the dam that contains fine soil particles.

However, it is the contention of this paper that expedient decisions in the construction and design of embankment dams ("on time, on budget") has also contributed to future dam safety issues. The Oxford dictionary defines expediency as "a regard for what is politic or advantageous rather than what is right or just". In the case of dams, expedient decision during construction with respect to encountered conditions, construction materials, and construction details can be the direct cause or source of future hazards.

This premise will be explored by considering several embankment dam case histories where there is evidence of poor or expedient engineering/technician decisions which have come to light decades after completion of the construction and which affect dam safety. It is noteworthy that the effects of the poor decisions will generally be borne by future generations of engineers who inherit the management or dam safety responsibility for the dams. It is pointed out that these concerns are not just of recent development, as one of the examples cited dates from 1850. It should also be appreciated that it is not just engineers and technicians who are responsible for such developments but also dam owners who, in their capacity of decision makers and financiers, are not blameless.

1.1 *Bilberry Dam failure: ~ 1850*

1.1.1 *Background*

The failure of Bilberry dam 150 years ago was one of the precursors that lead to the legislated dam safety situation that prevails in the UK today. Although an old case history, the events leading to the failure remain a vivid example of how things can go wrong because of various human factors.

1.1.2 *Failure*

The dam failed in 1852 with the flood wave killing 81 people and causing substantial economic damage. Sketches of the dam failure, and the subsequent damage it caused, were published in the Illustrated London News. Even today, these sketches present a chilling picture.

Technically, the initiation of the failure mechanism was caused by piping at the contact of the puddle clay with the foundation. This piping then developed into a sinkhole that extended upwards to the dam crest and allowed overtopping to develop. Binnie (1981) gives details of the construction and the failure mechanism.

1.1.3 *Expedient factors*

Interest here centers on the human factors, and there are several that contributed:

i. The dam was promoted by local businesses, but from the outset the funds raised were insufficient for the scale of the works.
   There was a spring found in the dam foundation once it had been excavated. However, rather than constructing a drainage culvert to accommodate it (as was the practice at that time), the Inspector (Falshaw) directed that puddle clay be placed over the spring in the apparent expectation that ongoing clay placement would force the clay into the spring and seal it. There seems to have been no review of this decision by the Engineer, possibly because their fees were seemingly not being paid for their services which were under dispute at the time. It is unclear whether the decision to place clay fill and not to construct a culvert was caused by an inexperienced inspector making a poor decision or whether it was decided that a culvert was an additional cost to the contract that could not be afforded.
ii. Leakage and settlement of the dam continued. Settlement was obvious occurring, amounting to some 3 m by about 1849 (three years before failure). Seepage was visible from the toe of the dam since about 1846 (six years before failure). Although the Owner could have lowered the spillway inlet to maintain freeboard, even with the restricted funding available, this lowering was not done by the Owners as it would have reduced the dam's value to the operating of the water mills for which it was constructed.

The points stated above are a litany of expedient decisions and clearly so; the appropriate technical response (to provide a culvert to capture and divert the spring water) was known. The cause of the dam failure was the result of human, decisions not a technical error. Arguably, the problems all stemmed from insufficient funding being available in the first place for the scale of works contemplated.

1.2 *Vancouver Island Dam: ~ 1950*

1.2.1 *Background*

We now move forward a hundred years from the early days of engineered dams to the era of modern geotechnical engineering. This dam was designed and constructed in the mid 1950s on Vancouver Island, British Columbia. A conventional embankment dam about 22 m high and 200 m crest length, it was constructed on bedrock using coarse sand and gravel shells, a transition zone (filter) and a thin clay core (a plan layout of the dam is shown on Figure 1). The current safety classification is as a 'high consequence' dam; this classification requires that dam be able to resist the maximum credible earthquake—in excess of 0.5 g.

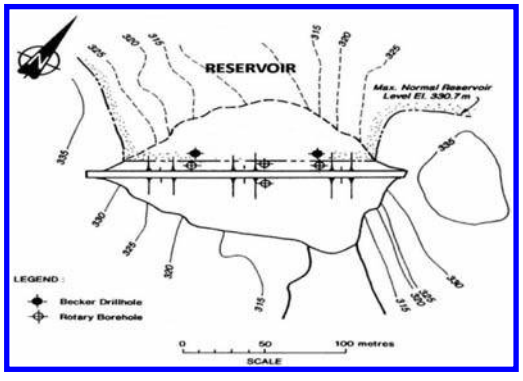

Figure 1. Plan of dam showing borehole locations.

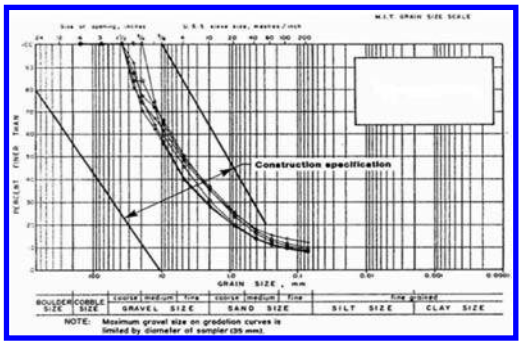

Figure 2. Typical grain size distribution curves for samples from the shell of the dam.

At the time of original construction earthquake engineering was in its infancy and if the design considered seismic issues at all (and there is no documentation that it did), consideration would almost certainly have been in pseudo static adopting an acceleration of 0.1 g. The dam was underdesigned when considering current standards and resistance requirements. An investigation was initiated to review the seismic withstand of the dam under current regulations.

The investigation in the late 1980s comprised boreholes drilled through the dam was carried out to investigate the construction conditions of the embankment fills. Standard and Becker Penetration Tests, Classifications tests on soil samples of the embankment fills were carried out as per a convential modern investigation. Soil samples were obtained. In addition, detailed construction records were reviewed.

The results of several grain size analyses carried out on representative samples of the shell fills are given on Figure 2. In general although this material would be barely within specification tolerance and the fills would contain a larger percentage of fine particles than desired making it more susceptible to water disturbance.

1.2.2 *Expedient factors*

The presence of loose layers (as shown on Figures 3 and 4) within a relatively modern engineered dam was puzzling: these loose layers must have been a result of something during construction. A review of the construction records for the dam was made. This review was enlightening—there had been (i) a labour strike and (ii) a forest fire closure just after construction commenced. From the timing of these events and documented construction progress, these events occurred very close to where the potentially liquefiable loose layer was located. After these construction incidents were over, construction resumed but was carried out predominantly in the months of September and October and thereafter ceased over winter until the following spring. Examination of the rainfall and weather records for the two months of September and October indicated that at the site experienced very wet conditions. The inspectors reports also indicated that the construction was frequently suspended due to bad weather over this period and in some cases the fill surfaces were wet and spongy. There was also concern at the time to achieve sufficient elevation to prevent overtopping during winter shutdown. It is postulated that the situation had developed to the point where construction staff accepted less than normal (and specified) compaction to allow sufficient construction for safety before the winter shutdown—a necessary response. This decision would have also allowed the project to continue on schedule and construction costs to be maintained.

The dam was remediated by flattening the slopes to provide adequate resistance under seismic conditions.

Given the absence of present concerns for earthquake resistance during the dam's construction, an expedient reduction in the compaction standard may have seemed reasonable at the time. However,

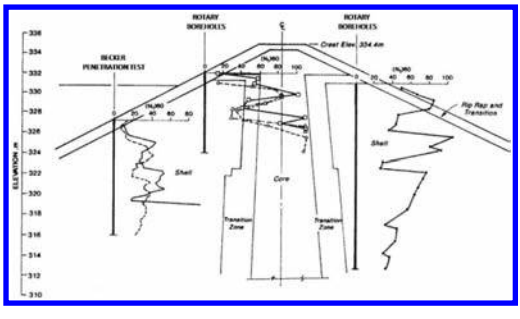

Figure 3. Schematic section through the DAM showing the computed $(N_1)_{60}$ values for the BPT and SPT boreholes in the sandy gravel shell materials.

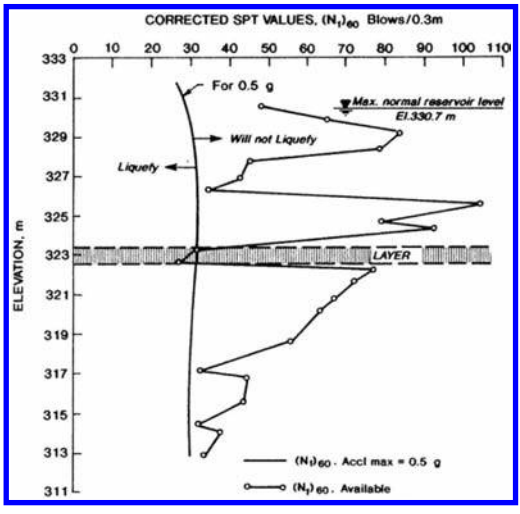

Figure 4. Plan of dam showing borehole locations.

no evidence of review of this decision by the engineer of record was found in the files. And, it is also true that additional resources could have been mobilized, at increased cost, to construct the fill more closely in accordance with the design. Like Bilberry dam, even though we are now in the modern engineering era some 100 years later, there is the suspicion on people making changes outside of a proper review process because of cost and schedule concerns.

1.3 *Southern Ontario Dam: ~ 1970*

1.3.1 *Background*

This dam was constructed in the early 1970's as a municipal water retaining structure. It was set in a glacial valley with a till-covered valley floor underlain by interbedded shale and limestone. This earthfill dam was about 20 m high and 300 m crest length, and was a conventional modern design with a sloping clay core, graded filters and compacted granular shells. Side slopes were conventional. The designers were aware of the pervious nature of the rocks beneath the till covering the valley floor and the dam included a grout curtain to cut-off seepage in the pervious strata by extending downwards to form a seal in an underlying shale stratum.

Seepage from bedrock was evident for a considerable distance downstream of the dam on first filling. However, as seepage was anticipated (despite the grouting) this was not a concern. There was little that changed in the following five years. But, at that time, seepage was now observed from the downstream face of the dam and the seepage was "dirty" with a substantial portion of the dam toe being "spongy". Not surprisingly, emergency lowering of the reservoir followed.

1.3.2 *Consequences*

The dam safety surveillance program caught the deterioration of the dam before it could develop into a piping failure. And, the loss of stored water, although inconvenient, was accommodated elsewhere. The actual consequence was an extensive investigation and remediation program, at substantial and unbudgeted cost.

1.3.3 *Expedient factors*

Although this dam safety incident did not develop into a failure, again there is, like the BC dam in the 1950's, the puzzling question of how a modern, engineered dam could have got into an incipient failure state. The investigation prior to remediation showed nothing than was not previously known. However, once grouting was chosen as the remediation method, construction records of the grouting carried out during construction were recovered and reviewed and, these records made the origin of the problem clear.

The grout curtain that was constructed extended across only the base of the valley, not the full width of the dam. It remains unclear whether that was the original design (although there are no geological factors as to why this would have been reasonable) or whether the curtain was shortened during construction for cost and/or schedule reasons. But, what was abundantly clear is that the largest grout takes were at the end of the curtain where it was terminated, and this area was upstream of the incipient piping that then developed. What appears to be construction expediency is the termination of the grout curtain where it stopped—any conscious engineer should have extended the curtain based on the ground response to grout injection. The worst ground was left untreated. One is left to wonder about the role of the dam owners financial resources.

Technically, there was one further factor in this dam safety incident. During the original site investigation, great difficulty was found in distinguishing the overlying till from the underlying shale because of the weathering in the shale. A plausible explanation is that seepage through the fissures in the shale after impoundment eroded the shale, increasing fissure apertures, and causing an accelerating situation. This danger appears unrecognized in the original design and construction. What was further rather remarkable is that after remedial grouting largely stopped that seepage, what appears to have been shale swelling resulted in a complete seal developing over the next two years.

1.4 *Northern BC dyke failure: ~ 1970*

1.4.1 *Background*

A sewage treatment facility in a northern community of British Columbia included dykes, the dykes being constructed to contain two anaerobic cells,

one aerobic cell and a large storage pond. The cells were rectangular construction with dyke heights of the order 3 to 4 m. However, although low by dam standards, the dykes are an interesting case history as they were modern, engineered soil structures and yet they failed.

The foundation conditions on which the dykes were constructed consisted of interlayered alluvial silts and sands. The dykes were constructed of excavated material from inside the cells and consisted predominantly of silty and sandy soils. The outside and inside slopes were 3 horizontal to 1 vertical and 4 horizontal to 1 vertical respectively. The dykes were constructed by conventional means and the soil fills were compacted with a sheepsfoot roller; the specified density was in general achieved and there were no issues raised about low compaction.

1.4.2 *The failure*

After completion of the pipes, the ponds were gradually filled with sewage. The effluent level in the aerobic cell increased gradually to within 0.3 m of design operating level at which time there was what was locally termed a "blowout" of the dykes at the location of the aerobic cell drain with complete failure of the dyke and loss of the effluent. Inspection of the failure indicated the following:

- The breach in the dyke had occurred along the drain pipe (see Photos A and B).
- The dyke breach exposures indicated that the dyke was comprised of compacted silty soils and the exposure in the dyke was nearly vertical.
- There was no evidence of saturation or seepage in the remaining dyke section.
- The pipe was intact but evidently bent downwards from its original predominantly horizontal alignment (see Photo C).
- The drain pipe had been connected to a larger outfall pie which had been constructed below original ground surface. This outfall pipe was pushed upwards and now was evident above the original ground surface (see Photo A).

Photo A

Photo B

Photo C

Complete reconstruction was of the dyke was necessary which set back the commissioning of the sewage system by several months and caused budget overruns for which no money was in reserve.

1.4.3 *Expedient factors*

The observations after the failure showed that failure was caused by one or more of the following:

- Internal erosion due to seepage along the decant pipe.
- Internal erosion due to seepage along or adjacent to the contact of the general dyke fill and the recompacted trench fill.
- Piping as a result of under seepage at the toe of the dyke acerbated by seepage concentration due to the installation of the pipe.

Seepage along the pipe appears to have received insufficient attention during design, with only provision of a small, nominal seepage collar. Quite how that got through a review process is an interesting question.

Although a defective design appears the starting point for the failure, this poor design was further compromised by actions at site. There are no records of fill being checked for compaction in the trench around the pipe and there is visual evidence that the contact of the dyke fill with the trench

fill was not well integrated or blended. It would appear that on site construction decisions were made, probably as a result of construction scheduling, that such small fills were minor and were not a major issue to the performance of the system. And, again, the records do not indicate ongoing review/supervision by the engineer of record.

2 DISCUSSION AND CONCLUSION

Issues caused by expediency arise three ways in the context of dams.

First, there is expediency at the design stage. Terzaghi reiterated in many of his addresses that, because of unknowns defining the conditions in a dam and/or its foundation, good engineering requires a fallback position in the event of untoward conditions developing (no more than encountered in other aspects of engineered construction where redundant load paths are included as a basic safely philosophy). And it is warranted with dams since most cases of catastrophic dam failure relate to a combination of events that each in themselves might have been a symptomatic problem but not of which were a sufficient hazard to cause rupture in themselves. Expediency arises when such features are omitted at the design stage to save on perceived costs and/or construction schedule. The southern Ontario dam described above might have been a case of this if the grout curtain was designed to only partly extend along the dam axis.

Second, there is expediency at the construction stage. As with all construction involving soil or rock, there are aspects of a dam's foundation that cannot be determined by any practical site investigation program. Inspection of conditions revealed during construction is an integral part of an engineered dam. But, this then leads to expedient choices if the implications of untoward conditions are glossed over or otherwise downgraded in their significance to avoid additional works from originally planned. A relate expediency is changing details and so forth during construction to reduce costs and/or remain on schedule. Both the Vancouver Island and Southern Ontario dams appear implicated in this.

Third, there is expediency by dam owners. This can range from proving too little resources at the investigation or design stage to cutting back on engineering effort during construction. Expedient behaviour of owners can also arises in lack of resources devoted to safe stewardship of the constructed dam, although this is becoming rare in industrialized countries because of dam safety legislation. One of the earliest example of such legislation traces in part to the Bilberry Dam failure described above, that case being a litany of expedient actions by the owner.

However, it seems to the authors that the engineering profession is not entirely blameless. In civil engineering, any dissociation of the engineer of record from at least a review role during construction must be seen as expedient cost saving that is likely to lead to detectable/correctable faults being unnecessarily incorporated in the constructed work. Nowadays the engineering profession, at least in North America, has accepted or has been forced to accept, the practice of 'design professionals' thereby with the possible exclusion of the designer from the construction. This doctrine in our opinion legitimizes bad practice and ultimately can lead to the development, in the case of dam engineering, dam safety issues at a later date.

Looking forward, it is noteworthy that dam safety assessment procedures, for example those of the Canadian Dam Association (CDA, 2005), focus on technical factors. Errors in the field of dam engineering will continue to occur because as Blockley (1980) succinctly states "all error is human error, because it is people who have to decide what to do, it is people who decide how it should be done and it is people who do it." All we can do is be aware of our limitations and learn from the past in the hope that we can attempt to anticipate and ameliorate future problems.

We suggest that an appreciation of the context of a dam's engineering and construction might alert engineers carrying out dam safety assessments to issues that perhaps should be anticipated—there should be an expansion of dam safety assessment protocols to include the context of a dam's construction in additions to currently measurable aspects of its performance.

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Effect of reservoir operation on the performance of concrete face rockfill dams with accessible plinth: The case of Undurraga Dam

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ABSTRACT: Rockfill dams with reinforced concrete facing leaned against an accessible and verticalized plinth are known to develop different types of seepage and destabilisation phenomena. Understanding the causes behind each particular case is fundamental to build effective predictive models based on instrumentation data, which can thereafter be applied to monitor the dyke structural safety and to justify decisions on reservoir operation and restoring works. Here we present the case of Undurraga Dam (Biscay, Spain), which has shown an increasing seepage through the embankment since its construction, in a trend only broken by several partially-successful cement grouting campaigns. Its evolution has been related with the amplitude and frequency of filling/emptying cycles in the reservoir, leading to quick alternations of hydraulic loading and relaxation over the plinth and provoking its downstream gyre. Taking into account these facts, a conceptual mathematical model has successfully been developed and fitted.

1 INTRODUCCIÓN

Una de las principales precauciones a considerar en el diseño de una presa de escollera con pantalla aguas arriba es la necesidad de evitar filtraciones a través del cuerpo del dique que conduzcan a fenómenos de erosión o asiento y a un eventual fallo (De Justo Alpañes & González Martínez, 1986). Para evitar dicho extremo, la ejecución de un plinto sobre el que apoye la pantalla y que, o bien penetre en el cimiento hasta alcanzar un substrato impermeable o bien sirva de base para inyecciones del terreno, se considera esencial (Murthy Challa, 2002)

Dicho plinto debe estar correctamente cimentado en el terreno, con una profundidad función de la calidad de la roca y su grado de fracturación (McDanield, 1987), habiéndose demostrado su validez en caso de empotramiento total (Sherard et al., 1963). En otros escenarios, se debe asegurar la correcta estabilidad del elemento (vuelco y deslizamiento) pudiendo llegar a requerirse anclajes, sobre todo si la altura de agua sobrepasa los 30 m (CNCGP, 1995), o bien la retirada de material del cimiento y su sustitución por hormigón, el ensanchamiento del elemento y la inyección del terreno, sobre todo si hay fallas o grietas importantes, rellenos de arcilla o parte de la roca es erosionable (Vallarino Cánovas del Castillo, 1991). De no ser así, pueden registrarse movimientos del plinto que provoquen la apertura de vías de filtración en su contacto con el terreno o incluso la apertura de fisuras en la pantalla anexa (Fernández Ordóñez & Alonso Franco, 1982) que comprometan la seguridad estructural de la presa.

El diseño usual de plintos prima la dimensión horizontal sobre la vertical, confiando su resistencia frente al deslizamiento a un conjunto de anclajes y al propio rozamiento con el terreno (Fell et al., 2005). Sin embargo, la posibilidad de incorporar al plinto una galería interior para inspección, drenaje y tratamiento del cimiento sin necesidad de vaciar el embalse y de dotarle de una doble finalidad de apoyo de pantalla y ataguía de obra, condujo en el pasado hacia diseños más verticalizados como el que se presenta en la Figura 1, que normalmente se ejecutaban mediante encofrado y se semi-empotraban en una zanja, apoyados sobre un bloque de cimentación de hormigón en masa y protegidos con rellenos de arcilla. La estabilidad de este tipo de elementos se confió inicialmente al empuje pasivo de la escollera del trasdós, aspecto que posteriormente se demostró erróneo, al requerir su activación movimientos importantes (Pérez Sáiz, 1989). Como consecuencia, este tipo de solución, profusamente aplicada en España y usual en Europa en presas de escollera en las décadas de 1970 y 1980, ha presentado patologías frecuentes (Utrillas Serrano & Sánchez Caro, 2006; Soriano Peña & Sánchez Caro, 1997).

La detección de un fallo del plinto de este tipo es compleja, por cuanto el incremento de la cuantía de las filtraciones a través del dique que llevaría

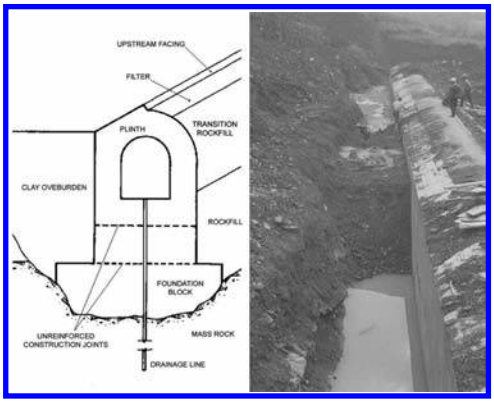

Figura 1. Sección típica del plinto de la presa de Undúrraga (izq.) Ejecución del relleno de arcilla aguas arriba (der.).

asociado puede obedecer a otras causas, como la rotura de la propia pantalla o un deficiente tratamiento del terreno de apoyo (Tancev, 2005). Agotados los ensayos directos, es más bien el análisis conjunto de sus movimientos (caso de disponer de auscultación a tal efecto) y su correlación con las filtraciones observadas, el que debe aportar información sobre el problema existente, sobre todo si no es posible interrumpir temporalmente el servicio.

Conocido el fenómeno, existen distintas opciones de reparación, desde el anclaje del elemento, la consolidación de la escollera en trasdós o el sellado de la fisura plinto-cimiento (Alonso Franco & Soriano Peña, 1988), bien tras el vaciado del embalse o actuando desde la galería de inspección del plinto si es que ésta existe. En el caso de que las actuaciones anteriores no sean factibles o su eficacia sea únicamente parcial, las exigencias de seguridad a satisfacer para este tipo de presas aconsejan disponer de algún tipo de herramienta predictiva de la evolución futura de los caudales de filtración y/o movimientos del plinto, que permitan acotar la zona de comportamiento normal o previsible, de la extraordinaria. De esta manera, se pueden detectar con claridad episodios de crecimiento anormalmente rápido que aconsejen la declaración de una situación de emergencia, con las consecuencias en cuanto a actuaciones de corrección, vigilancia y comunicación que esto llevaría aparejado (ASCE, 2000). El desarrollo de dicha herramienta debe partir necesariamente de un extenso registro de variables de auscultación que permitan caracterizar de forma exhaustiva el comportamiento histórico de la presa, y que posteriormente alimenten un modelo matemático a ajustar (Díez Cascón-Sagrado, 2003)

Este modelo puede ser exclusivamente estadístico, en función de variables como la carga hidráulica o la temperatura (MMA, 2001). Sin embargo, si la repuesta de la estructura es compleja o depende de varias causas interrelacionadas entre sí, puede requerirse la formulación y ajuste de un modelo de tipo conceptual, basado en ecuaciones representativas del fenómeno observado.

Este es el caso de la presa de Undurraga (Biscay, Spain), de materiales sueltos y pantalla de hormigón, que constituye un elemento clave para el suministro al Área Metropolitana de Bilbao, lo que implica la necesidad de adoptar unas condiciones de seguridad muy estrictas para evitar un eventual desabastecimiento.

La presa, desde su puesta en carga, ha manifestado un problema recurrente de filtraciones a través del cuerpo del dique, cuya cuantía ha aumentado progresivamente en el tiempo a pesar de las sucesivas campañas de inyección efectuadas (en la actualidad alcanza los 100 l/s), constituyendo la mayor amenaza existente frente a su estabilidad a largo plazo. Las causas de esta patología pueden ser varias, si bien la existencia de un plinto visitable potencialmente inestable (ver Fig. 1), que ha requerido obras de refuerzo en el pasado, parece apuntar hacia la apertura progresiva de una posible vía de filtración preferente bajo esta estructura como consecuencia de su movilización, que comunicaría el embalse con algún accidente geológico o con una zona de cimiento muy fracturado.

En este sentido, del análisis exhaustivo de los datos de auscultación se ha podido identificar una relación entre el ritmo de progresión de las filtraciones y la forma de explotación del embalse a través del efecto de ésta sobre el giro del plinto hacia aguas abajo. Considerando estos tres factores, se ha confeccionado y ajustado con éxito un modelo predictivo compatible incluso con la ejecución de trabajos puntuales de inyección. Este modelo ha permitido establecer el ritmo esperable de progresión del fenómeno y dotar al equipo de explotación de los argumentos necesarios para fundamentar la toma de decisiones sobre la forma de explotación del embalse más aconsejable.

2 LA PRESA DE UNDÚRRAGA

2.1 *Descripción del aprovechamiento*

La presa de Undurraga, situada sobre el río Arratia, en el término municipal de Zeanuri (Biscay—Cuenca Cantábrica), entró en servicio en el año 1974 como contraembalse del Salto Hidroeléctrico de Barazar, del que recibe las aguas trasvasadas desde el embalse de Urrunaga (Cuenca Ebro) (Ibisate González de Matauko, 2005). Con un volumen útil de 1,92 hm^3 a cota de Máximo Embalse Normal, constituye a su vez el origen de prácticamente el 75% del suministro en alta al Área Metropolitana de Bilbao, actuando como depósito de regulación del sistema general de abastecimiento del Consorcio de Aguas

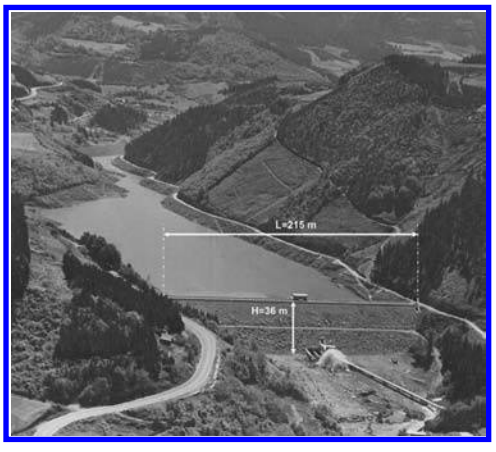

Figura 2. Vista aérea de al presa y embalse de Undúrraga.

de Bilbao-Bizkaia, lo que implica importantes variaciones diarias de nivel (Puerta, 1990).

La tipología de presa es de escollera caliza (talud aguas arriba y aguas abajo de 1,4H:1V y 1,75H:1V respectivamente) con una altura sobre cimientos de 36 m. La planta es recta, con una longitud de coronación de 215 m. Como elemento de impermeabilización dispone de una pantalla de hormigón armado sobre el talud aguas arriba, apoyada en un plinto perimetral al que se adosan dos galerías de pie de tapiz en ambas laderas. La pantalla está compuesta por placas de 4 m de ancho separadas por juntas impermeables sobre capa granular. En su pie lleva dos juntas perimetrales separadas 3 m a modo de biela para permitir asientos diferenciales. La presa cuenta con un cuerpo central de hormigón en el que se sitúan el aliviadero de superficie, las tomas y los desagües de fondo y suplementarios, y que divide la estructura en 2 desde el punto de vista deformacional.

2.2 *Reseña histórica*

La presa de Undúrraga fue construida entre los años 1970 y 1974 como variante de un proyecto inicial cuyas obras fueron aprobadas en 1965 y que contemplaba la ejecución de una presa de gravedad y planta curva aguas abajo del actual emplazamiento, considerado posteriormente inviable por problemas de deslizamiento de laderas.

Al objeto de mejorar la cimentación, durante las obras y en margen derecha se realizaron inyecciones de mortero o arcilla en una primera fase y de lechada de cemento en una segunda a 1 kg/cm^2 de presión, mediante taladros de 15–25 m. En el fondo del valle se inyectó lechada de cemento únicamente en el sector de argilitas y areniscas.

La presa se puso en explotación en 1974 alcanzando el MEN en enero de 1975, oscilando posteriormente entre las cotas 206 y 210 m. Desde el primer momento se observaron filtraciones crecientes que aparecían en la galería del plinto perimetral a través de los taladros de drenaje y aguas abajo aflorando por el pie de la escollera. Los drenes con mayor caudal fueron rellenados con arena o granalla.

Para evaluar este fenómeno, se realizó en 1984 un estudio teórico del comportamiento de cada uno de los bloques del plinto, que demostró la incapacidad del diseño existente para soportar el empuje del agua, lo que habría provocado el desplazamiento del elemento hacia aguas abajo para apoyarse en la pantalla y escollera, y la consecuente apertura de vías de filtración. Además se dedujo que el plinto no estaba lejos de deslizar según la junta de construcción con el bloque de cimentación, lo que podría acentuar todavía más el problema. En consecuencia, se procedió en 1991 a desarrollar diversos trabajos de estabilización, que incluyeron el cosido del plinto al cimiento mediante anclajes pasivos, la inyección de la escollera en su trasdós, el sellado de fisuras y la rehabilitación del drenaje.

Una vez finalizadas estas obras, se constató únicamente un descenso de las filtraciones del orden de un 25%, si bien se consiguió asegurar la estabilidad del plinto. Sin embargo, a partir de 1994 se inició un programa de auscultación hidráulica intensiva que puso de manifiesto de nuevo el crecimiento progresivo e incluso aumentado de las filtraciones. A la vista de ello, el Consorcio de Aguas de Bilbao-Bizkaia redactó en 2001 un proyecto de localización y corrección de filtraciones que se ejecutó en verano de 2002, consistente en el sellado mediante lechada de cemento a varias profundidades bajo los bloques 1, 2, 3 y central, y que no tuvo los resultados esperados. No obstante, la valoración de la efectividad de este tipo de tratamientos ha sido siempre problemática por la variabilidad de la explotación.

En 2004 y ante la evolución creciente de las filtraciones, se realizaron 3 nuevas perforaciones en los bloques 1 y central, consiguiéndose en una de ellas una reducción apreciable del caudal aforado. En los mismos bloques pero en este caso mediante resinas epoxi y acuarreactivas, se ejecutaron en 2007 9 taladros que no tuvieron apenas influencia.

2.3 *Datos de auscultación*

Inicialmente la auscultación de la presa de Undurraga se confío tanto al control topográfico de desplazamientos de coronación y pantalla como al seguimiento de los caudales de filtración y subpresiones en la pantalla de drenaje de la galería del plinto y en el exterior. Sin embargo, tras constatar de la posible inestabilidad del plinto de la presa, la Confederación Hidrográfica del Norte, antigua titular del aprovechamiento, decidió acometer

una serie de mejoras en la instrumentación que permitiesen controlar en detalle los posibles desplazamientos de dicho elemento. Como consecuencia, el sistema de auscultación actual de la presa comprende las siguientes variables además de las usuales de explotación (nivel de embalse y caudales de entrada y salida) y meteorológica (precipitación y temperatura):

- Filtraciones en galerías (6 aforadores en cunetas)
- Filtraciones exteriores (3 totalizadores)
- Filtraciones y subpresiones en drenes (98 taladros)
- Niveles freáticos aguas abajo (3 catas)
- Giro del plinto (27 clinómetros)
- Deformación del plinto (44 pares medidores de juntas entre bloques en 3 dimensiones)

Figura 3. Aforador de filtraciones VE-I en pie de presa y margen izquierda (izq.) y clinómetro C-1 en Bloque 1 (der.).

- Apertura de fisura plinto-cimiento (8 extensómetros de doble varilla)
- Desplazamientos del cuerpo de presa (17 hitos en bateolas de coronación y 40 en dos hileras sobre la pantalla de hormigón)

2.4 *Evolución de filtraciones y giro del plinto*

En la Figura 4 se presenta la evolución temporal de las filtraciones en el aforador exterior izquierdo (VE-I), que es el que recoge la mayor parte del caudal filtrado a través del cuerpo de presa. Como puede apreciarse, el incremento de las filtraciones ha sido notable desde los 10 l/s iniciales hasta alcanzar los 100 l/s actuales. Sin embargo, este crecimiento no ha sido constante y ha sufrido incluso cambios de tendencia coincidiendo con algunos hitos reseñables de la historia de la presa, pudiéndose distinguir en este sentido:

1. Las obras de refuerzo del plinto de 1991 que rebajaron algo su cuantía pero que dieron paso a una época de elevado ritmo de crecimiento
2. Cambio en la explotación del embalse hacia ciclos de llenado/vaciado más cortos en 1999
3. Campañas de inyección de 2002 (con cierta estabilización de caudales), 2004 (la más efectiva, con una reducción de unos 20 l/s) y 2007 (con apenas influencia)

En lo que respecta al giro del plinto, todos los bloques han seguido un patrón similar (en la Fig. 6 por simplicidad únicamente se representa el del bloque 1: C-1) con un ligero vuelco hacia aguas abajo y dos tendencias de crecimiento bien diferenciadas: una más

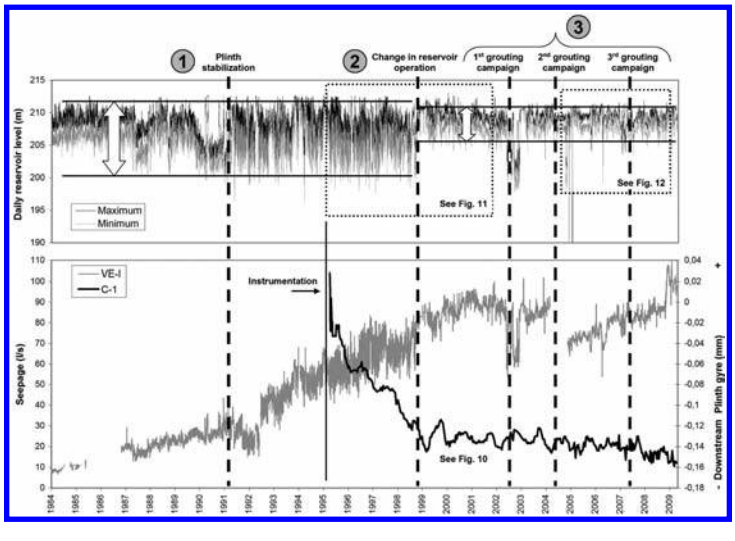

Figura 4. Evolución temporal de la filtración observada en el aforador VE-I y giro del plinto medido en el clinómetro C-1 (abajo). Variación del nivel de embalse máximo y mínimo diario en el mismo periodo (arriba).

acentuada de 1995 a 1999 y otra más moderada desde entonces y hasta la actualidad.

En la Figura 4, en la que se han representado los niveles también máximos y mínimos diarios del embalse, puede observarse cómo, dejando a un a lado los efectos puntuales de las campañas de inyección, el cambio de explotación del embalse desde ciclos de llenado/vaciado más largos (2–3 días) a otros más cortos (1 día) que se produjo al inicio 1999, afectó simultáneamente y en el mismo sentido al ritmo de crecimiento del giro del plinto y de las filtraciones a través del cuerpo de presa, reduciéndolos significativamente en ambos casos. Este hecho apunta a la existencia de algún tipo de relación de causalidad, cuya identificación y caracterización a partir del análisis detallado de los registros de auscultación es objeto del presente trabajo.

2.5 *Geología*

El conocimiento de la geología del emplazamiento se considera fundamental para el análisis de cualquier problema de filtraciones en presas. En este caso y según la cartografía regional, el sustrato es en general impermeable, formado por lutitas y areniscas del Tránsito Jurásico-Cretácico (facies Purbeck) en disposición monoclinal con buzamientos moderados hacia la margen izquierda. Sobre el mismo hay una cobertera variable de suelos aluviales y coluviales. Esta geología teórica contrasta con algunos datos de los Proyectos Constructivos original y modificado (1968 y 1970) y con la información únicamente gráfica aportada por el archivo de las obras, que describe áreas calcificadas y de la que se puede interpretar la existencia de zonas verticalizadas junto a otras zonas prácticamente horizontales, así como huecos y numerosas fracturas (ver Figs. 5 y 6)

La margen derecha sería más homogénea litológicamente y presentaría menor fracturación, si bien los espesores de suelo serían superiores. La zona central y margen izquierda, en cambio, presentarían materiales al menos potencialmente

Figura 5. Zona de falla bajo el cuerpo central de la presa. Destaca la alta fracturación de lado derecho.

Figura 6. Cimentación bajo el Bloque 3. Detalle de cavidad rellena de arcilla y bolos (izq.) y zona con discordancia estructural y hundimiento/falla (der.).

carstificables y una mayor fracturación, tal y como quedó de manifiesto en los taladros de investigación efectuados en 2007 bajo el bloque 1, lo que apoyaría la hipótesis de que existió un saneo insuficiente de la superficie de cimentación durante la construcción.

Por otro lado, los datos fotográficos documentan la presencia de fracturas bajo el cuerpo central y bloques 1 y 3 de la margen izquierda. Estas fracturas parecían favorecer una cierta permeabilidad local. De hecho, identificadas las profundidades con mayor caudal asociado en las perforaciones ejecutadas durante la campaña de inyección del año 2002, se puede interpretar la existencia de una fractura subvertical en estos bloques que habría repercutido sobre dos niveles más sensibles al incremento de la permeabilidad por fracturación (probablemente paquetes areniscosos). Integrando todos los datos disponibles, esta fractura sería oblicua a la cerrada y de traza aproximada N-S, una orientación que podría relacionarla incluso con una potente falla identificada en el macizo rocoso de margen izquierda unos 200 m aguas arriba de la propia presa. Esta fractura podría explicar la existencia de una importante filtración profunda, si bien no justificaría por si sola el ritmo de crecimiento observado.

2.6 *Investigación del fenómeno*

Varios han sido los ensayos efectuados con el objetivo de determinar las causas de las filtraciones observadas y en consecuencia, establecer las mejores actuaciones para su corrección.

Así, durante los trabajos de estabilización del plinto de 1991 y ante la escasa efectividad de los mismos en la reducción de los caudales aforados en el exterior, se perforaron 20 taladros en los bloques 0 y 1 en busca de una posible vía de filtración profunda, resultando infructuosos. Se pensó entonces en una posible rotura de la pantalla. A tal efecto se realizaron pruebas de llenado-vaciado y el

vertido de sacos de cemento, concluyéndose en la posible existencia de una fisura por la junta plinto-pantalla de los bloques 1 y 3.

En 2002 y antes de proceder a las labores de impermeabilización, se perforaron 32 taladros para aforo del caudal fluyente a diferentes profundidades, constatándose al menos dos zonas con cierta continuidad bajo los bloques 1 y central, que podrían puntualmente ponerse en contacto con la junta plinto-cimiento (ver Fig. 7), lo que también resultaría coherente con el hecho de que durante la fase de inyecciones en los bloques 1, 3 y central, normalmente se detectase con mayor frecuencia la aparición de mortero de cemento en el aforador exterior izquierdo al aumentar la presión o reducir la profundidad de obturación. También se observó en ocasiones burbujeo en el embalse, lo que se achacó a una conexión directa de éste con los estratos permeables. Por otro lado, durante la campaña de 2007 se volvió a detectar caudal en el contacto plinto-cimiento pero sobre todo en torno a los 20 m de profundidad.

Más recientemente y coincidiendo con los trabajos del Análisis General de la Seguridad de la presa desarrollados en 2009, se decidió acometer una serie de trabajos de investigación que permitieran mejorar la eficacia de futuras inyecciones. Así, en primer lugar se efectuó una inspección subacuática de la parte visible de la pantalla (la no aterrada). En ella no se encontraron defectos reseñables en las juntas y únicamente se localizó lo que podía interpretarse como una cavidad en el fango, entre los bloques central y 1 y a la altura de la junta horizontal, sin poder llegar a determinarse la existencia de succión ni que la cavidad fuese resultado de una posible filtración.

Se aplicaron posteriormente distintos métodos geofísicos de reconocimiento: potencial espontáneo (ver Fig. 8), potencial inducido y tomografía eléctrica. Las medidas realizadas identificaron dos zonas anómalas en el vaso con posible continuidad a través de la presa: una bajo el cuerpo central y otra por margen izquierda, interpretables como zonas de fractura. Por el contrario, no se detectaron posibles grietas en la pantalla de hormigón.

Por último, se desarrolló una campaña de detección de fugas por trazadores salinos (salmuera) vertidos en el embalse en puntos seleccionados, registrándose la respuesta en términos de conductividad en aforadores exteriores e interiores. Como resultado de la aplicación de esta técnica, se pudo acotar la zona problemática entre los bloques 5 y 13 de la margen izquierda, al ser los que mayor y más rápida respuesta provocaban en el aforador exterior izquierdo. Esta observación coincide con lo indicado por la geofísica y señala a una zona hasta ahora escasamente tratada en las campañas de inyección acometidas.

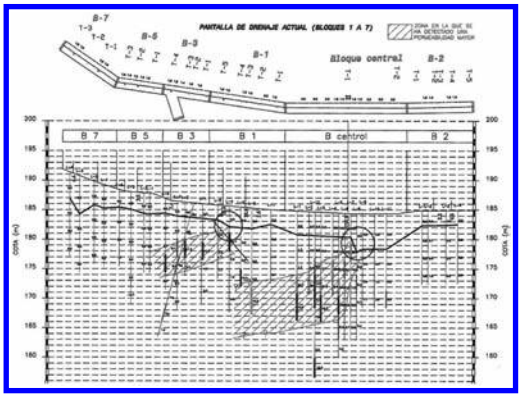

Figura 7. Zonas de mayor permeabilidad bajo Bloques 1, 3 y central. Se señalan los contactos con la junta plinto-cimiento.

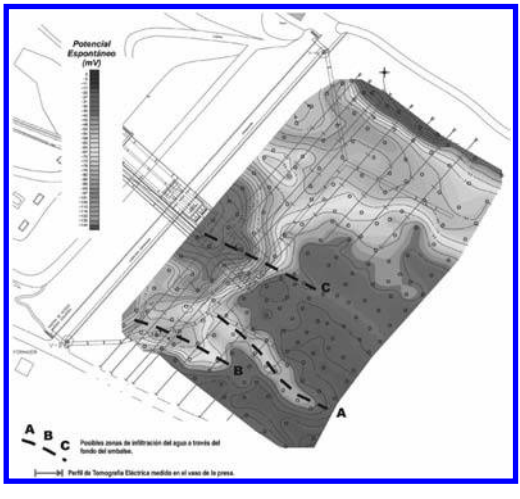

Figura 8. Valores de autopotencial (mV) aguas arriba de la presa de Undúrraga. Posibles fallas en raya discontinua.

3 MODELO PREDICTIVO

Como ha quedado de manifiesto anteriormente, las posibles causas de la filtración detectada a través del cuerpo de presa son variadas y su localización es todavía incierta. En este contexto se considera que la información que puede aportar la auscultación histórica de la presa es relevante para entender la patología detectada, pero sobre todo para estimar su evolución futura. Para ello, se procede a continuación al planteamiento y ajuste de un modelo conceptual al giro del plinto (manifestado en el clinómetro C-1) y a las filtraciones a través del cuerpo de presa (según registros del aforador VE-I).

3.1 *Principios básicos*

La idea fundamental que subyace en este análisis es la confirmación del efecto que la forma de explotación del embalse, con una variación rápida de nivel y una sucesión casi diaria de ascensos y descensos, habría tenido en el pasado sobre la estabilidad del plinto y, como resultado de ésta, sobre la evolución de las filtraciones.

Dado que los anclajes de estabilización del plinto que se implantaron en 1991 son pasivos, el elemento puede girar por acción del empuje hidrostático, hecho que queda de manifiesto en los registros disponibles en los clinómetros. El comportamiento del cimiento bajo el plinto, caracterizado por una fracturación elevada, puede razonablemente suponerse plástico y, en consecuencia, es posible que parte del giro resultante de una operación de llenado no se recupere completamente tras el vaciado. La acumulación de estos giros residuales sería la responsable del movimiento detectado (Olivier, 2006). Además de esta tendencia a largo plazo, en los valores del giro del plinto se identifica también una onda térmica de amplitud reducida, que sería coherente con la observada a su vez en las juntas entre bloques.

Esta deformación progresiva habría permitido la apertura de vías de filtración bajo el plinto que, o bien conectarían directamente con el aforador exterior a través de la escollera de presa o con la capa superficial y más alterada del cimiento, o bien pondrían en comunicación el embalse con alguna de las fracturas geológicas profundas que discurren bajo el dique. En el primer caso, la cuantía de las filtraciones debería responder a un determinado coeficiente de desagüe que multiplicaría a un área de paso creciente en función del aumento del giro y/o los propios ciclos de variación del nivel de embalse, y a la raíz cuadrada de la altura de carga. En el segundo caso, en cambio, las filtraciones serían el resultado de una determinada permeabilidad multiplicada por el área de paso y por el gradiente hidráulico. Lógicamente, también son factibles soluciones intermedias. Por otro lado y como consecuencia de la circulación de agua a través del cuerpo de presa/cimiento, no es descartable la existencia de fenómenos de lavado de finos que habrían incrementado sucesivamente el área de paso.

Para verificar la hipótesis anterior y dada la disyuntiva planteada, el procedimiento operativo para discernir el mecanismo real de la filtración a partir de los registros históricos de auscultación se ha basado en el desarrollo y ajuste de un modelo conceptual al giro del plinto (representado por el clinómetro C-1) y al caudal observado en el aforador exterior izquierdo. Dicho modelo debe partir de los datos de niveles de embalse y temperaturas (acciones externas) y, mediante ecuaciones sencillas en función de los conceptos antes expresados, representar fielmente la realidad de las variables controladas (efectos). El grado de validez del modelo, es decir, su bondad de ajuste, se asume como justificación de las hipótesis propuestas y, en cualquier caso, debe permitir discernir las más plausibles de aquellas con escaso fundamento físico.

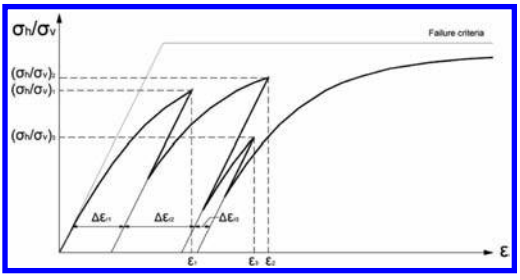

Figura 9. Esquema supuesto de comportamiento elastoplástico y deformación residual del cimiento del plinto.

Una vez ajustados los modelos, su empleo como predictores del comportamiento futuro de la presa resulta evidente. En el caso del giro del plinto, la calibración o ajuste se ha efectuado para todo el periodo de medidas, es decir, de 1995 a la actualidad. Sin embargo, en el caso de las filtraciones, la calibración se ha limitado al periodo 1995–2002 que es el que transcurre hasta la primera campaña de inyecciones, ya que este hito lógicamente debió modificar las condiciones existente del flujo. Por otro lado, como paso previo antes de su aplicación, se ha efectuado también una validación temporal de noviembre de 2004 en adelante, que es la que permite su aplicación predictiva (las inyecciones del año 2007 apenas tuvieron influencia en el caudal).

3.2 *Formulación*

A efectos prácticos se ha supuesto que el embalse desciende en un día si la cota mínima registrada coincide con la máxima del día siguiente, mientras que asciende si es la máxima la que coincide con la mínima del día siguiente. En otros casos, se considera que existe ascenso y descenso en el mismo día. La cuantía de estos movimientos se obtiene de la forma:

$$R_t = H_{\max,t+1} - H_{\min,t}$$
$$F_t = H_{\max,t} - H_{\min,t+1} \quad (1)$$

donde R_t = ascenso en el día t; F_t = descenso en el día t; $H_{\max,t}$ = cota máxima de embalse del día t; $H_{\text{minx},t}$ = cota mínima de embalse del día t.

Admitido que el giro resultante del empuje hidrostático es proporcional al cuadrado de la variación de nivel, lo que supone la aplicación de una relación tensión-deformación de tipo parabólico para el cimiento del plinto el giro residual

tras un ciclo completo de llenado y vaciado puede expresarse como:

$$\theta_{r,t+1} = \theta_t^+ - \theta_t^- = A \cdot R_t^2 - B \cdot F_t^2 \quad (2)$$

donde $\theta_{r,t+1}$ = giro residual; θ_t^+ = giro hacia aguas abajo tras ascenso; θ_t^- = giro hacia aguas arriba tras descenso; A y B = constantes a ajustar.

Por consiguiente, el giro del plinto en un momento dado será el resultado de la acumulación de los giros residuales en los intervalos anteriores más un giro inicial asociado al origen de las mediciones. Adicionalmente, existe un efecto térmico que se supone lineal y aplicado a la diferencia entre la temperatura ambiental 4 meses antes de un determinado instante y la temperatura media a lo largo de la serie, de valor, 14,1°C. Este desfase entre acción y efecto térmico es usual en presas y es consecuencia de la inercia térmica de los materiales (Agulló Fite et al., 1997):

$$\theta_t = \theta_0 + \sum_{t=1}^{n} \theta_{r,t} + C \cdot \left(T_{t-4m} - 14{,}1\right) \quad (3)$$

donde θ_t = giro en instante t; θ_0 = giro inicial; C = constante; T_{t-4m} = temperatura ambiente 4 meses antes del instante t.

En lo que respecta a la filtración, su cuantía se considera proporcional a la altura de carga hidráulica sobre el aforador (con cota absoluta de 187 m) elevada a un exponente a definir.

$$Q_t = K_t \cdot (H_{emb,t} - 187)^{\alpha} \quad (4)$$

donde Q_t = caudal de filtración en el día t; K_t es la constante de proporcionalidad del día t; $H_{emb,t}$ = cota de embalse en el día t registrada las 8 horas; α = exponente variable entre 0,5 y 1.

La constante de proporcionalidad, entendida como el producto de un determinado coeficiente de desagüe por el área de paso, es a su vez variable en el tiempo en función tanto del giro incremental del plinto (asociado a la apertura de la vía de filtración) como de la acumulación sucesiva de ciclos de llenado/vaciado (asociado a un efecto progresivo de lavado de la zona de circulación de agua) de la forma:

$$K_t = D + E \cdot \theta_{r,t}^{\beta} \cdot \sum_{i=1}^{t} \left(H_{\max,i} - H_{\min,i}\right) \quad (5)$$

donde D y E = constantes; $\theta_{r,t}$ = giro relativo registrado en instante t, β = exponente, $H_{\max,i}$ y $H_{\min,i}$ = cota máxima y mínima de embalse en el día i.

3.3 *Ajuste*

El valor de los parámetros que controlan el comportamiento de los modelos planteados en el apartado anterior no puede conocerse a priori. Su obtención, en cambio, debe confiarse a un proceso de calibración por comparación con los datos de auscultación registrados, de forma que se alcance un ajuste satisfactorio con la realidad. En la medida que esto sea posible, podrá validarse su empleo a futuro así como confirmarse las hipótesis propuestas en su confección. De igual modo, su cuantía podrá interpretarse en términos físicos, aportando claves para el conocimiento del fenómeno.

El proceso de ajuste se ha confiado en este caso al algoritmo de tipo genético de Nelder & Mead (1965) que permite la minimización de una determinada función objetivo para un conjunto de parámetros limitados a un rango de validez. Este método utiliza un *simplex* o conjunto de posibles juegos de valores de los parámetros (para un modelo de n parámetros, el *simplex* se compone de n + 1 juegos de parámetros), evolucionándolo hasta alcanzar un vértice en el que la función objetivo sea mínima. Para ello, en primer lugar se estima el juego de parámetros pésimo y óptimo dentro del *simplex* y, en segundo lugar, se sustituye el juego pésimo mediante reflexión, expansión o contracción respecto del resto, o mediante reducción hacia el juego óptimo. Este algoritmo se ha implementado en una rutina Visual Basic. Como función objetivo se ha adoptado el coeficiente de eficiencia o R^2 (Nash & Sutcliffe, 1970) empleado usualmente para medir la bondad de modelos conceptuales dado que involucra la estandarización de la varianza residual y su valor esperado no cambia con la longitud del registro o la magnitud de la variable simulada. Un perfecto ajuste sugiere un valor igual a 1, cuando el valor es cero indica que el modelo no es mejor si se compara con uno de una sola variable (por ejemplo un valor medio) y valores negativos indican que el modelo se comporta peor. En general, se consideran como excelentes los ajustes con un R^2 mayor que 0,8 y aceptables si superan 0,6.

Operando de esta forma, se alcanza un mejor ajuste para el giro del plinto para:

- $\theta_0 = -0{,}015$
- $A = -9{,}5 \cdot 10^{-6}$
- $B = -8{,}2 \cdot 10^{-6}$
- $C = 0{,}001$

consiguiendo un R^2 de 0,95 (ver Fig. 10) lo que supone un resultado adecuado y valida la justificación del fenómeno propuesta. El plinto, por tanto, estaría girando debido a la acumulación de deformaciones residuales como consecuencia de ciclos sucesivos de carga/descarga. Dado el valor de las contantes A y B, el giro residual se estima en torno al 13,5% del asociado al ascenso del embalse, indicando un comportamiento prácticamente plástico. Sobre esta tendencia a largo plazo se superpondría un efecto térmico estacional.

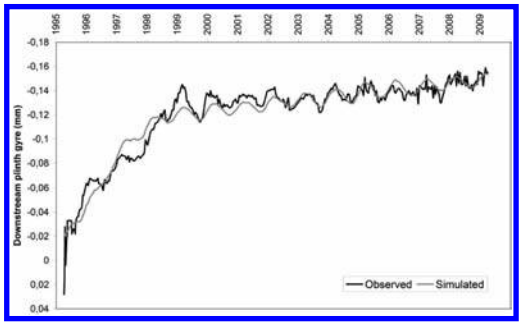

Figura 10. Ajuste del modelo del giro del plinto.

En el caso de la filtración, la calibración del modelo da como resultado:

- D = 6,9
- E = 0,059
- $\alpha = 0{,}72$
- $\beta = 1{,}35$

consiguiendo un R^2 de 0,92 (ver Fig. 11), lo que confirma la veracidad de las hipótesis planteadas. En este caso, el valor del exponente α indicaría que el fenómeno de filtración sería mixto, existiendo una componente estructural y otra geológica. Esta conclusión se ve también refrendada por el análisis de los recientes datos de auscultación continua del aforador VE-I. Así, desde enero de 2010 se dispone de una lectura automática mediante sensor de ultrasonidos del caudal registrado en este punto, pudiéndose comparar con el valor del nivel de embalse que también se encuentra automatizado. Existe en general una correspondencia muy buena entre ambas variables, pudiéndose establecer un ajuste lineal con un error de nos más de 5 l/s para periodos temporales cortos en los que el efecto de crecimiento de la filtración a largo plazo puede despreciarse. Sin embargo, el mejor ajuste se consigue para un desfase entre nivel y filtración de 40 minutos ($R^2 = 0{,}935$), que sería el tiempo de respuesta medio y que apuntaría hacia velocidades de flujo de varios centímetros por segundo.

3.4 *Validación*

La bondad de un modelo matemático para representar una realidad física debe descansar no solo en una adecuada calibración sino también en una validación de su comportamiento para otros escenarios temporales diferentes del ajustado. En el caso de las filtraciones registradas en el aforador VE-I, se dispone del histórico de caudales a partir del año 2002, si bien la situación en este caso no es directamente comparable, dado que las campañas de inyección efectuadas desde entonces han podido alterar parcialmente el mecanismo de flujo.

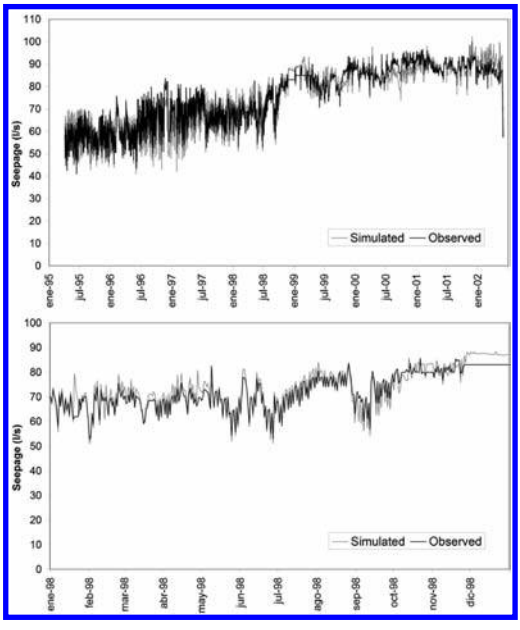

Figura 11. Ajuste general del modelo de filtraciones de 1995 a 2002 (arriba) y detalle de año 1998 (arriba). En este periodo se produce el cambio de explotación.

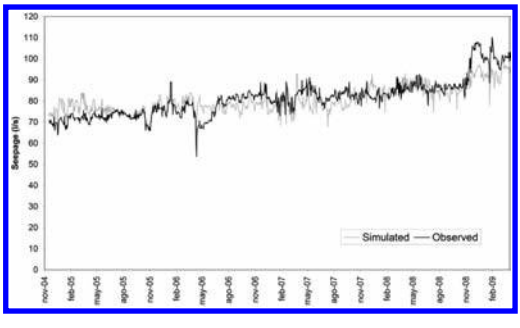

Figura 12. Validación del modelo de filtraciones de 2004 a 2009.

No obstante, si se pretende emplear el modelo confeccionado para la predicción de caudales a futuro, resulta imprescindible realizar este ejercicio. En este sentido, en la Figura 11 se presenta la comparación entre valores simulados y observados de noviembre de 2004, una vez finalizada la segunda campaña de inyecciones, en adelante. Para la simulación se ha respetado el valor de los parámetros del modelo calibrado salvo el de la constante de inicialización D que se ha reducido hasta 3,9 para tener en cuenta el sellado parcial de la vía de filtración como consecuencia de las inyecciones. De esta forma se obtiene un ajuste aceptable ($R^2 = 0{,}65$, ver Fig. 12) que consigue modelizar correctamente la tendencia creciente de la filtración.

4 CONCLUSIONES

La presa de Undúrraga, de escollera con pantalla de hormigón, constituye un claro exponente de las patologías asociadas a la ejecución de un plinto visitable y verticalizado parcialmente empotrado en el terreno. La posibilidad de un fallo estructural de este elemento requirió en el pasado obras de estabilización que no pudieron, sin embargo, solventar el problema de filtraciones crecientes a través del cuerpo de presa que continúa en la actualidad. La identificación de las causas que motivan este fenómeno y su ubicación exacta es compleja, lo que ha dificultado el éxito de las tres campañas de impermeabilización desarrolladas hasta la fecha.

No obstante, de la observación de los datos históricos de auscultación se puede concluir en la existencia de una relación entre la forma de explotación del embalse, que se caracteriza por la rápida sucesión de ciclos de llenado y vaciado, y la evolución no solo del caudal de filtración sino también del giro del plinto hacia aguas abajo. En este escenario, resulta plausible suponer que se estaría activando algún tipo de mecanismo que relaciona el movimiento del plinto con el aumento de la circulación de agua en el cimiento. La naturaleza de este mecanismo se ha investigado en el presente trabajo mediante la confección y ajuste satisfactorio de un modelo conceptual a estas dos variables.

Por un lado, el giro del plinto se derivaría de una deformación plástica del cimiento que implicaría la existencia de un giro residual tras cada ciclo completo de llenado vaciado (carga/descarga). El ritmo de esta deformación se habría ralentizado considerablemente tras el cambio de explotación hacia ciclos más cortos acontecido en 1999.

Por otro lado, la filtración por margen izquierda sería proporcional a la altura de carga hidráulica elevada a un exponente intermedio entre el de desagüe por un orificio artificial y el de circulación por un medio poroso. La velocidad del flujo estimada sería también coherente con esta interpretación. Además, la constante de proporcionalidad aumentaría con el tiempo en función del giro registrado en el plinto y los ciclos de llenado/vaciado. Se observa también una ralentización del crecimiento tras el cambio de explotación de 1999. Como explicación a este comportamiento se propone una apertura gradual de la junta plinto-cimiento que habría puesto en contacto el embalse con alguna estructura de falla bajo la presa o con el propio terreno de apoyo, que se encuentra altamente fracturado. El paso del tiempo habría propiciado también un lavado progresivo del área de flujo. La existencia de una geología más compleja de la inicialmente establecida en proyecto y el consecuente saneo insuficiente del cimiento durante la construcción contribuirían al problema. En consecuencia, las futuras campañas de impermeabilización deben centrarse en el sellado tanto de la junta plinto-cimiento como de posibles vías de filtración profundas.

Además de facilitar la explicación de las patologías detectadas en la presa, la disponibilidad de los dos modelos ajustados permite la predicción de la evolución más probable del giro del plinto y de la filtración así como la evaluación de la influencia en ellas de la forma de explotación del embalse. En este sentido, de mantenerse el tipo de explotación actual y no ejecutarse nuevas inyecciones, la cuantía de la filtración registrada en el aforador VE-I en el año 2015 sería de 120 l/s mientras que una vuelta a ciclos de llenado/vaciado más largos supondría 30 l/s adicionales, siempre y cuando la deformación plástica del cimiento continuase. Un análisis similar puede efectuarse para el giro del plinto.

Con los modelos confeccionados se puede, por tanto, definir unos umbrales que de ser superados en los próximos años significarían un cambio no previsto de tendencia como resultado de un agravamiento del fenómeno. En tal caso, se debería declarar una situación de emergencia que desencadenase estudios exhaustivos y/o actuaciones urgentes de corrección.

Se demuestra, en consecuencia, que un análisis detallado y razonado de los datos históricos de auscultación permite mejorar substancialmente el conocimiento del comportamiento de la presa y aumentar su seguridad global.

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An evaluation and comparison of rock-fill dam behavior with instrumentation data during first impounding

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ABSTRACT: Permanent control of embankment dam stability during construction, the first impounding and also during utilization of the project, is a significant issue. Instrumentation of such dams which monitor their behavior plays an important role in the dam stability. Regarding that an earth dam is a continuous structure with non-linear behavior, finite element method and appropriate soil modeling could be employed for non-linear analysis. In the present research, based on the instrumentation data of a rock-fill dam, the induced stresses and also pore water pressure during first year impounding have been compared with analytical results. In order to achieve adequate accuracy 3 sections of dam which being fully instrumented, were considered. Two soil models including Mohr-Coulomb and Hardening models were selected for stress-strain analysis. The results specified that Hardening model is more capable of predicting the dam behavior and determining the dam stability during construction and afterwards, especially during first impounding.

1 INTRODUCTION

The limitation of water resources in the world has caused to implement various methods to get maximum efficiency from the large dams. The short and long term stability is the most vital issues for engineers. The importance of monitoring programs for dam safety is widely accepted, since it is the best method to control the stability process of large dams [US Army Corps of Engineers. 1995]. Structural monitoring has become an integral part of the safety concepts for complicated structures such as earth dams. Hereby, the main goal is to recognize damages, defects in structural safety or external threats to safety, such that necessary measures can be taken in due time and without any reduction in safety [Myers & Stateler, 2008]. The concept for the structural monitoring of dams is based on safety assessments.

The use of instrumentation as part of dam safety programs is growing as the technology of instrumentation and ease of use improves. Instrumentation can be used to implement a monitoring program that provides more comprehensive and timelier information regarding the on-going performance of the dam.

The first impounding phase is probably the time in the life of the dam when performance instrumentation monitoring activities are most critical. As the reservoir is filled, the seepage resistance of the dam, foundation, abutments, and reservoir rim is being tested for the first time. Induced stresses and pore pressure in the core and the of dam shall be recorded together with factors such as water level that may influence the measuring [Norwegian Water Resources and Energy Directorate. 2004]. In embankment dams the progress of filling will cause induction and development of considerable pore pressure within the core of dam. If the developed pore pressure during construction has not been dissipated to some stages, it may double after the first impounding, leading to hydraulic fracturing of the dam [Hosseiny & Ahmadi, 2005].

Here, in the present research the stability of the Tabarak earth dam, located in the North East of Iran, is controlled by processing and analyzing induced stresses and developed pore water pressure during the first of impounding.

2 SPECIFICATIONS OF THE DAM

Tabark dam is constructed on Tabark River, located in the North-East of Iran. It is an earth fill dam with a central and vertical clayey core. The dam has a maximum construction height of around 74 m from the foundation base. The crest is 198 m long and 10 m wide with an elevation of 1513.5 m from the mean sea level. The core of the dam has a slope of 1H:5V with maximum and minimum widths of 37.5 m and 5 m, respectively. Figure 1 illustrates a general view of the dam. The main role of this dam which its operation launched in October 2007 is to provide drinking water for Quchan city.

Also shown in Figure 2 is a typical instrumented section of Tabark dam. All elevations in this Figure

Figure 1. A general view of Tabarak dam.

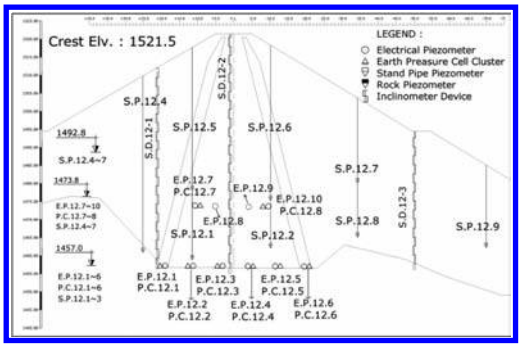

Figure 2. A general section of instrumentation.

Table 1. List of instruments installed in Tabark dam.

Type of instrument	Location	Quantity
Earth piezometer	Clayey core	44
Rock piezometer	Foundation	12
Stand pipe piezometer	Core and shell	30
Cell pressure	Clayey core	24
Extensometer	Abutment	2
In place inclinometer	Core and shell	9
Settlement plate recorder	Core and shell	9

are given in meters from the mean see level [Mahab Ghods Consulting Engineers. 2005].

3 LAYOUT OF THE DAM INSTRUMENTATION

Various types of instruments such as electrical piezometers, electrical pressure cells, standpipe piezometers, inclinometers, settlement gauges, and accelerometers were installed throughout the dam. These instruments were installed at different elevations in 3 monitoring sections (Figure 1). All main instruments, listed in Table 1, are installed in three sections, namely A-A, B-B and C-C [Mahab Ghods Consulting Engineers. 2005].

Here in this study, variations of pore water pressure and soil stresses in the clayey core of dam in all above three sections were investigated. The selected instruments for this research are located at the level of 1457 in the sections A-A and B-B, and at the level of 1448 in the sections C-C.

In order to measure induced stresses in all three directions and also calculate major principal stresses and shear stresses [Dunnicliff, 1993], a cluster of 5 pressure cells at each level was considered. In each cluster 5 pressure cells were installed in the following directions:

- Horizontal (to measure vertical stress)
- Vertical (perpendicular to the longitudinal dam axis to measure horizontal stress)
- Vertical (parallel to the longitudinal dam axis to measure horizontal stress)
- Inclined (parallel and 45 degree relative to the longitudinal dam)
- Inclined (perpendicular and 45 degree relative to the longitudinal dam)

4 NUMERICAL MODELING

Since the dam is constructed in wide valley and its behavior is adequately close to plain strain condition [Potts & Zdravkovic, 2001], the dam was modeled by Plaxis2D software which is a finite element method base [Brinkgreve & Vermeer, 2001]. This software uses different soil models including Mohr-coulomb and hardening models. The Mohr-coulomb model which is an elastic perfectly plastic model with a fixed yield surface requires routine and general soil parameters. In contrast to the above model the yield surface of a hardening model is not fixed in principal stress space and expands due to plastic strain.

This model is based on hyperbolic model [Duncan & Chang, 1970] and uses more complicated soil parameters. All the soil parameters which are commonly used for dam analysis (based on Mohr-coulomb and Hardening models) are listed in Table 2.

Table 2. Mechanical soil parameters of Tabark dam.

Material parameters	c	E_{ref}	ϕ	ν	γ_{wet}	γ_{sat}
	(kN/m^2)		(o)	-----	(kN/m^3)	
Clayey core	25	2181	29	0.35	19	19.5
Shell	0.3	30000	43	0.35	21	22
Filter (US)	0.3	40000	41	0.30	21	22
Filter (DS)	0.3	40000	39	0.35	21	22
Drainage	1	40000	41	0.35	21	22
Backfill (US)	0.3	3000	20	0.3	18.5	20
Backfill (DS)	20	3000	43	0.3	21	22
Rock foundation	150	2E + 07	27	0.3	22	23

Both unsaturated and saturated flow through porous media under steady and transient conditions may be modeled with this software. Pore water pressures and also induced stresses at sections AA, BB and CC during the first impounding and for 1 year steady state were evaluated.

5 EVALUATION OF MEASURED STRESS AND PORE WATER PRESSURE

Since the instruments installed at the level of 1457 for both sections A-A and B-B, are identical and also due to the limited pages of the paper, the instruments at the levels of 1457 and 1448 in sections A-A and C-C respectively are investigated.

5.1 *Stresses*

As mentioned earlier the induced stresses within the clayey core are measured using cell pressures. The processed data pertaining to horizontal, vertical and 45 degree stresses are illustrated in Figures 3 to 8 (Sections AA and BB).

The comparison of measured and calculated values indicates the proper operation of dam during first year of impounding. The observed good agreement between instruments data and calculated stresses, indicate that the mechanical soil parameters are accurately selected. Also the figures clearly show that the Hardening model is more capable of the prediction of dam behavior in comparison with Mohr-coulomb model.

It is well known that vibrating wire cell pressures are only able to measure stress perpendicular to the plane of cell, i.e. no shear or principal stresses can be measured using such these devices. If the cell pressures, however, are installed in a cluster

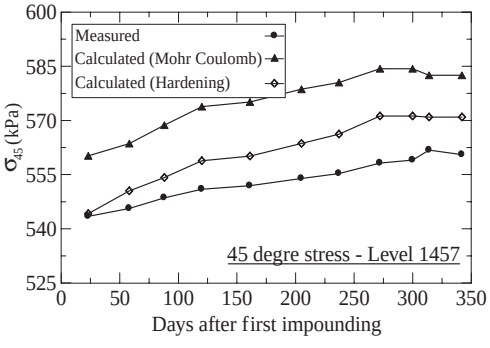

Figure 5. 45 degree stress (Level 1457, Section AA).

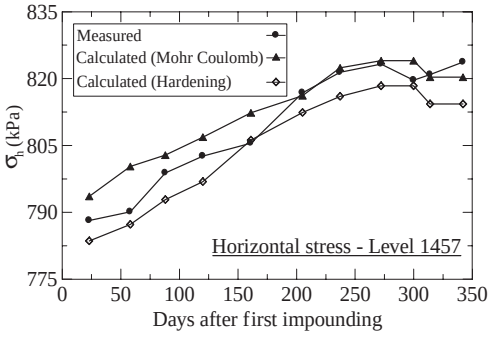

Figure 3. Horizontal stress (Level 1457, Section AA).

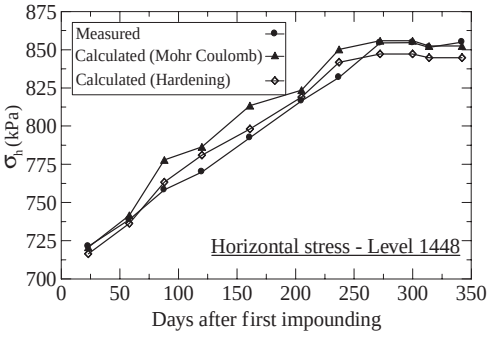

Figure 6. Horizontal stress (Level 1448, Section BB).

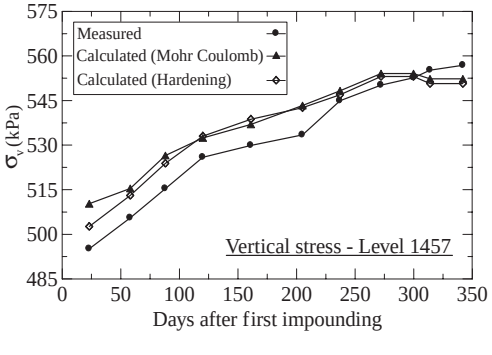

Figure 4. Vertical stress (Level 1457, Section AA).

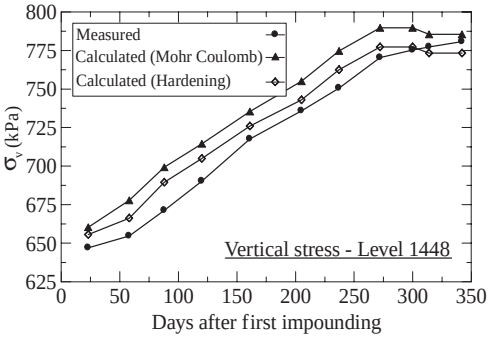

Figure 7. Vertical stress (Level 1448, Section BB).

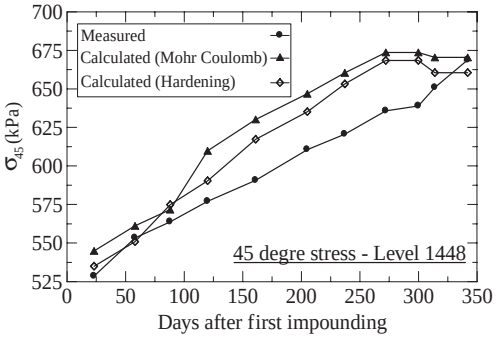

Figure 8. 45 degree stress (Level 1448, Section BB).

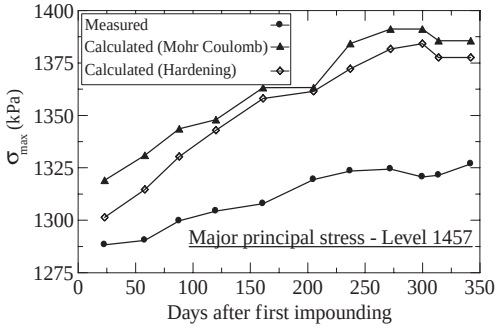

Figure 9. Major principal stress (Level 1457, Section AA).

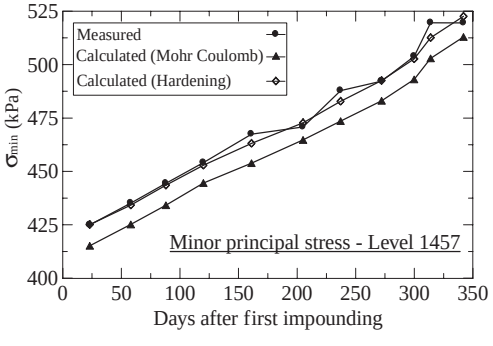

Figure 10. Minor principal stress (Level 1457, Section AA).

at different angles, it is possible to determine the induced principals and shear stresses, using classic elasticity theory [Ugural & Fenster, 1984]. Due the limited pages of the paper only the principal stresses are shown (Figures 9 to 12).

Again the comparison of measured and calculated values indicates the proper operation of dam during first year of impounding.

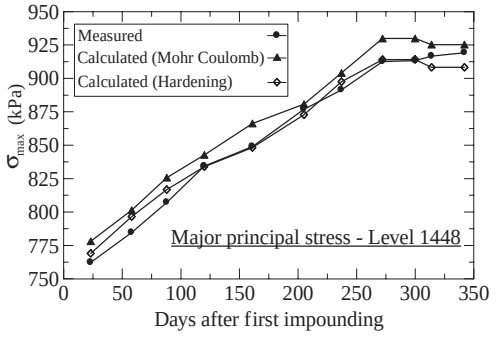

Figure 11. Major principal stress (Level 1448, Section BB).

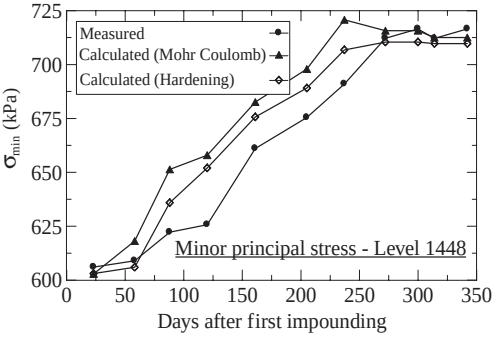

Figure 12. Minor principal stress (Level 1448, Section BB).

5.2 *Pore water pressure*

The pore pressures developed within the clayey core of dam in sections AA and BB at two different levels during the first year of impounding are processed and plotted in Figures 13 and 14. The data used here are according to the piezometers installed in the cluster located at level of 1457 and 1448, located in the central of the dam core.

Also shown in Figure 15 is the numerical analysis of pore water pressure distribution when the level water reservoir remained constant and the steady state could be considered.

As can be seen the upstream side of the core has been affected by the impounding while the downstream side has not been affected remarkably.

The other point here is the variations of pore pressure within the clayey core, when the no changes in the reservoir water level is occurred and it remains constant during a relatively long period (say about 3 month). This means that the pore pressure variations in the core are not affected by the reservoir water level variations. Figures 16 and 17 illustrate the variation of R_u with the reservoir filling process.

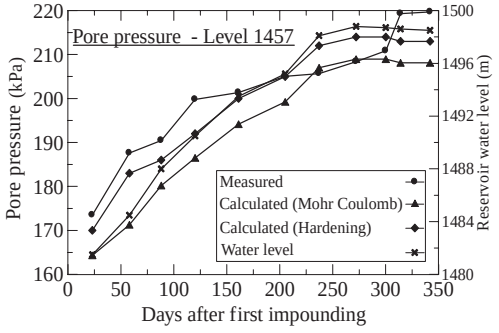

Figure 13. Pore pressure distribution (Level 1457, Section AA).

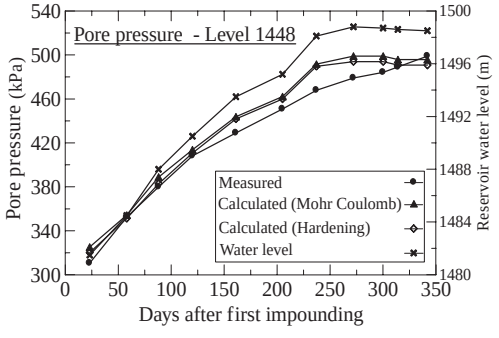

Figure 14. Pore pressure distribution (Level 1448, Section BB).

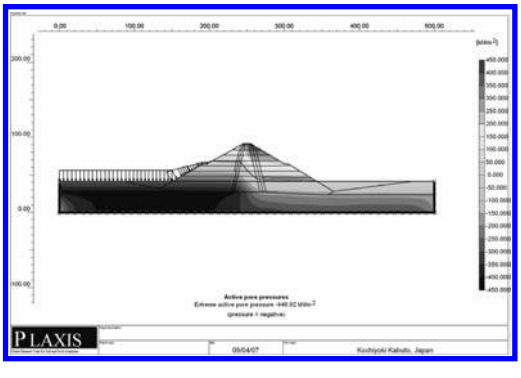

Figure 15. A cross section of pore pressure distribution.

In earth dams, in the other hand, due to differential settlement between various parts and settlement of central core compared to shells, the arching phenomenon may be seen. As a result, the part having more vertical deformation would transfer some amounts of its weight to the stiffer section. This conveyance which is called load

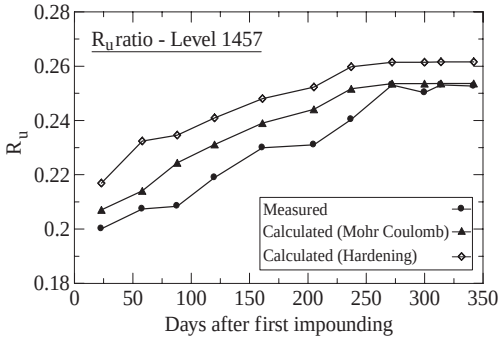

Figure 16. Variations of R_u with the reservoir filling process (Level 1457, Section AA).

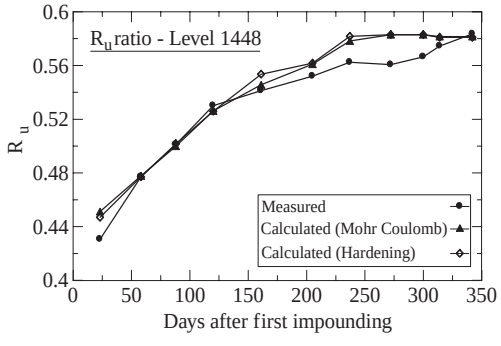

Figure 17. Variations of R_u with the reservoir filling process (Level 1448, Section BB).

transferring occurs because of arising shear stress at interfaces of two regions with different material and deformability. Therefore, reduction in the amount of vertical and horizontal stresses at lower parts of core happen which, in its turn, increase the possibility of fracturing. In other words, the more the core settles compared to shells, the greater the shear stress resistance develops at interfaces. Eventually, while the marginal parts of the core are maintained by shear stresses, discreteness of upper section from lower one may happen.

Here, the arching ratio is defined as the ratio of pore pressure at a point in the clayey core to the vertical stress at the same point. When the first stages of impounding commences, the core becomes gradually saturated, and this ratio (due to the pore pressure increasing) starts growing up.

Figures 18 and 19 illustrate the variations of arching ratio after the first impounding started. As can be seen this ratio is less than 1 during the first year of impounding, indicating acceptable safety factor against hydraulic fracturing.

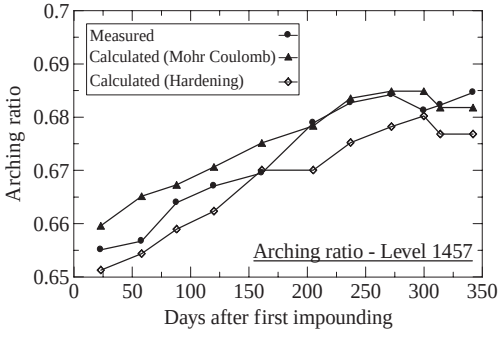

Figure 18. Variations of arching ratio with time (Level 1457, Section AA).

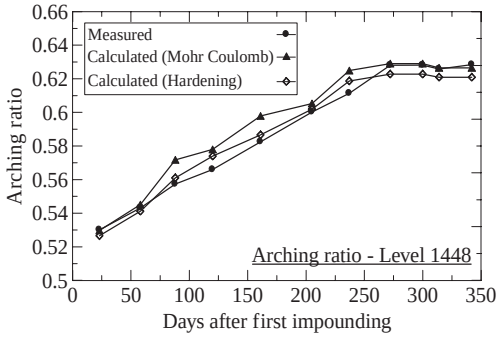

Figure 19. Variations of arching ratio with time (Level 1448, Section BB).

6 CONCLUSION

The Tabarak dam surveillance and monitoring during the first year of impounding was carried out by recording and processing instruments data. The numerical finite element model of dam was subsequently prepared and the dam analysis was performed using Mohr-coulomb and Hardening models. In the present research pore pressure development and induced stresses in the main and critical sections of an earth dam due to first stages of impounding has been investigated. The collected instrumentation data were then compared with numerical analysis. The results indicate the proper operation of dam during first year of impounding. The observed good agreement between instruments data and calculated stresses shows that the mechanical soil parameters are correctly selected. It can be finally concluded that Hardening model is more capable of the prediction of dam behavior in comparison with Mohr-coulomb model.

ACKNOWLEDGEMENTS

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Measured data control in concrete dams by back analysis

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ABSTRACT: In direct analysis of concrete dam, the geometry, the water head, uplift and all loadings are determined in such a way that the stress and strain are provided. In back analysis, deformation characteristics such as displacement, settlement, rotation and so on which have been measured by instruments used as an input data to detect the behavior of concrete dams. In two concrete dams in Iran, the back analysis was carried out. The measurements included: water level, temperature, and deformation. The curve of changes of water surface, the curves of concrete and water temperature were drawn. The deformations were drawn in all blocks and levels. The amounts of water level were less than predicated. The deformation was more than the calculated amount for design and the concrete temperature was less than the calculated limit. The main reason of the occurred drought was related to this period of measurement.

1 INTRODUCTION

In direct analysis of concrete dam, the geometry, location and elevation of water head gravity, uplift and all loadings are determined in such a way that the stress and strain are provided.

In back analysis, deformation characteristics such as horizontal displacement, settlement, rotation and so on which have been measured by instruments used as an input data to detect the behavior of concrete dams (Adriane, 2004).

Back-analysis is an approach commonly used in dam engineering to estimate strength and deformation parameters in-situ. This approach is popular because there are limitations in the use of laboratory and in-situ test results to accurately characterize dam strengths. Numerous studies have demonstrated the use of back-analysis techniques for the determination of dam behavior (Adriane, 2004).

2 MEASURED DATA IN CONCRETE DAM

2.1 *Instrumentation in concrete dam*

To do the back analysis, it is required to obtain the data related to measurement instrumentations in dam in a five or ten year period. These instruments are divided into two categories. The installed instrumentation in abutments and foundation also the installed instrumentation in dam body.

The instrumentation and their locations in dam body and its abutments are shown in Figure 1.

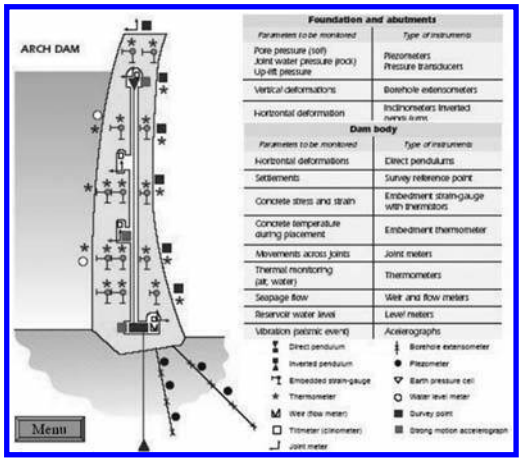

Figure 1. The instrumentation and their locations in a concrete dam (Kardeh dam report, 2003).

2.2 *Measurements in concrete dam*

The measurements include: water level, water temperature and concrete temperature, and all the deformation measurements were done by pendulum, and clinometers.

The curve of changes of water surface in terms of weather temperature and also the curves of concrete and water temperature in various blocks and levels were drawn. The deformations were drawn in all blocks and levels. The back analysis calculations in two concrete dams in Iran was carried out based on the measurement data in a ten year period.

3 KARDEH DAM

3.1 *Location and elevation of dam*

The Kardeh dam is located on the Kardeh River at the North-East of Iran in Khorasan County. The type of dam is double arch concrete with 67 meters height and 26×10^6 m^3 reservoir capacities (Figure 2). Figure 3 shows the plan and the profile of Kardeh dam (Kardeh dam report, 2003).

3.2 *Water level, weather and water reservoir temperature*

In Kardeh Dam, the water level in the first years of measurement (1994–2003) because of drought has abnormal fluctuations. But from 1996 to 2002 the highest water level occurred in the middle of May and the lowest water level took place at the end of October (Kardeh dam report, 2003).

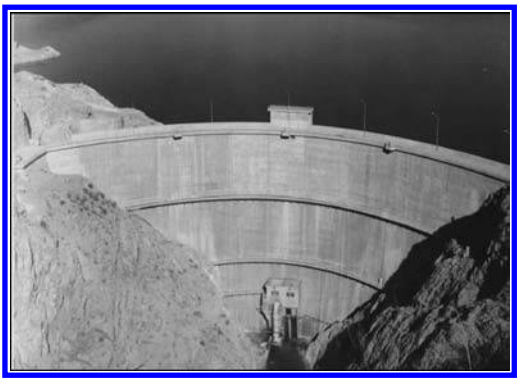

Figure 2. Kardeh concrete arch dam in north of Iran.

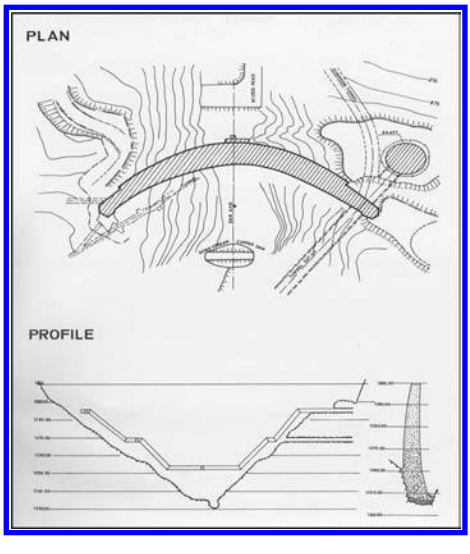

Figure 3. The plan and the profile of Kardeh dam (Kardeh dam report, 2003).

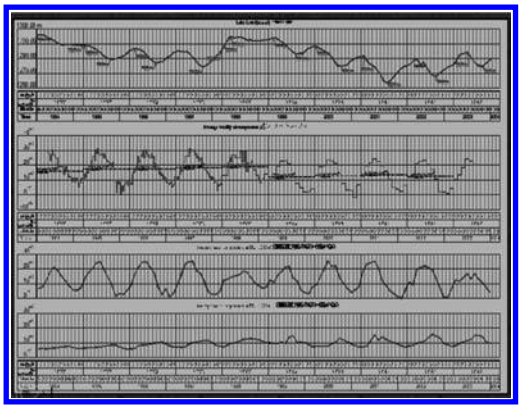

Figure 4. Water level, weather and water reservoir temperature.

The highest water temperature was about 20°C which occurred at the end of September. Regarding to the highest weather temperature which took place in the middle of August. There is a 1.5 months phase difference between the water temperature and weather temperature.

The main reason for this situation is that the weather of this region was arid during this decade of measurement Figure 4.

3.3 *Concrete body temperature*

Changes in the temperature of concrete can cause cracking. As the concrete gets hotter, it expands; as concrete gets cooler, it contracts. This is similar for many other materials too. If the concrete wants to expand or contract but can not, it will most likely crack. For example, the concrete near the foundation of a dam is restricted by the foundation there, the concrete can not expand or contract if it needs to and this might result in cracking. The main purposes of temperature measurement are as follow.

First, there will be some serious temperature goings-on during construction. Heat is a major result of the reaction between cement, water, and aggregate to form concrete. The bigger the structure, the more heat that's going to be generated. As the concrete cools, it may experience "shrinkage" some water evaporates and the concrete gets a little smaller. If shrinkage is not controlled, cracks can form.

Too much cooling to combat the heat of hydration could lead to problems though. If the concrete is cooled too much, it will have to warm back up to reach equilibrium, resulting in expansion and possibly cracking.

There may be seasonal difficulties during construction: the foundation of a dam may be poured in the fall and then pouring is stopped until after the winter and the top of the concrete block may be exposed to cold temperatures and want to contract but the bottom of the concrete block is restrained by the foundation.

In Kardeh Dam the concrete temperature measurements have been done for various levels in different blocks Figure 5. These measurements have been taken place in a ten year time period from 1994 to 2003. The temperature of concrete body in each block and each level has been measured in three layers, the upstream, middle and downstream. The average temperature in concrete body has been 15°C in the upstream layer 17°C in the middle layer and 14°C in the down stream layer in winter season. The more distance downward from the crest, the less the temperature of concrete body obtain. The temperature changes aren't considerable and the highest temperature of the middle layer of concrete body has been occurred in winter, and the downstream layer depends on the environment temperature and the upstream layer relies on the water temperature. The temperature changes of concrete body from the middle layer toward the foundation level showed an obvious decrease of temperature.

The temperature of the upstream and the middle layers of concrete body at average level and foundation depended on the temperature of the water and its fluctuations in summer and winter is very little.

The maximum temperature of concrete body in the middle layer occurs at the beginning of winter season and shows that in relation to the maximum weather temperature which generally happens in the middle of summer, there is a 4.5 month phase delay.

The temperature of concrete body in the downstream layer at average level and foundation depended on the weather temperature.

The temperature of concrete body in the upstream and the downstream layer at the crest level relied on the temperature of weather.

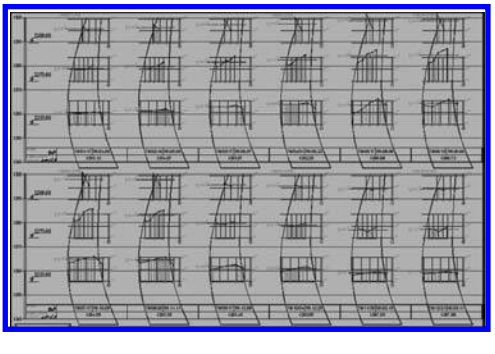

Figure 5. Concrete temperature for various levels in Kardeh dam.

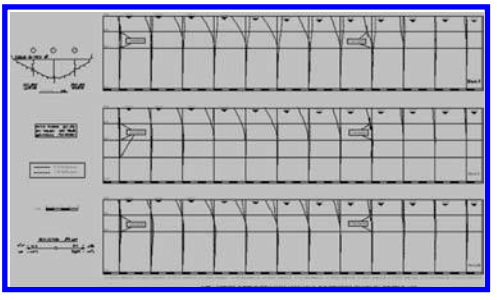

Figure 6. Tangible and radial displacement for various levels in Kardeh dam.

Table 1. Maximum displacement in various years in Kardeh dam.

Block 11		Block 8		
D/S	U/S	D/S	U/S	Year
2.20	–6.85	0.85	–10.40	1994
7.95	–7.55	–1.4	–12.65	1995
–0.30	–8.27	0.35	–	1996
2.65	–7.80	–1.0	–13.30	1997
–1.25	–11.6	–1.7	–14.75	1998
–1.40	–8.20	–2.4	–12.35	1999
–2.05	–9.45	–3.1	–14.25	2000
–1.00	–9.05	–2.5	–14.45	2001
–0.25	–9.15	–3.1	–15.05	2002
0.15	–9.55	–2.5	–18.55	2003
–2.85	–9.85	–4.8	–15.35	2004

In back analysis of dam, the phase delay of the temperature of concrete body with weather obtained 4.5 months while, according to the analysis of dam during the design period, this phase delay was estimated about 3 months.

Since, the construction of dam was finished about 20 years ago the temperature of the concrete mass is not due to the setting of concrete.

3.4 *Pendulum measurement*

Pendulum measurements and their locations in Kardeh dam are shown below. These measurements which are related to a ten-year period (1994–2003) in Kardeh Dam have been shown in the graphs of Figure 6.

4 TOROGH DAM

4.1 *Location and elevation of dam*

The Torogh dam is located on the Torogh River at the North-East of Iran in Khorasan County.

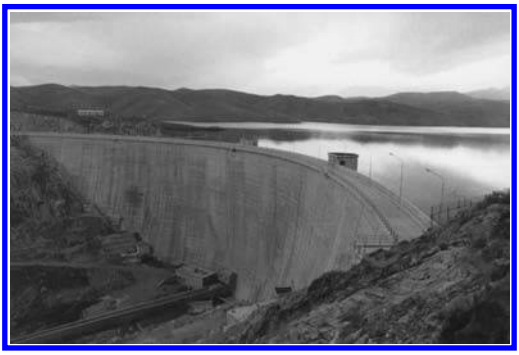

Figure 7. Torogh concrete arch dam in north of Iran.

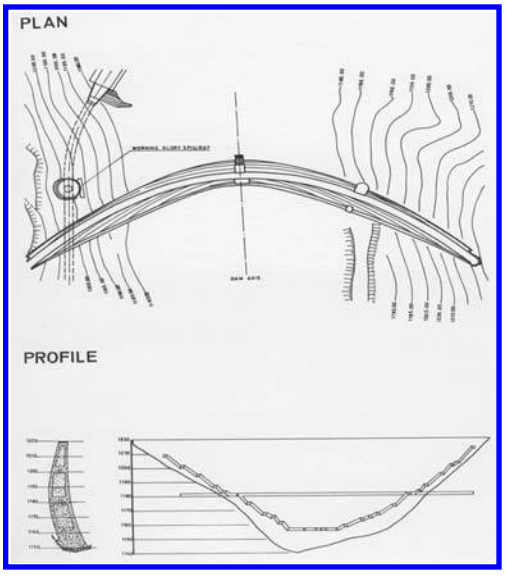

Figure 8. The plan and the profile of Torogh dam (Torogh dam report, 2003).

The type of dam is double arch concrete with 75 meters height and 40×10^6 m^3 reservoir capacities (Figure 7). Figure 8 shows the plan and the profile of Torogh dam (Torogh dam report, 2003).

4.2 *Water level, weather and water reservoir temperature*

In Torogh Dam, the water level in the first years of measurement (1994–2003) because of drought has abnormal fluctuations. But from 1996 to 2002 the highest water level occurred in the middle of May and the lowest water level took place at the end of September (Torogh dam report, 2003).

The highest water temperature was about 20°C which occurred at the end of September. Regarding to the highest weather temperature which took place in the middle of August.

Water level, weather and water reservoir temperature in Torogh Dam are shown in Figure 9.

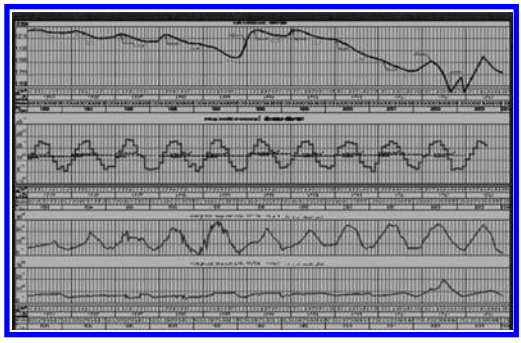

Figure 9. Water level, weather and water reservoir temperature.

4.3 *Concrete body temperature*

If the concrete wants to expand or contract but can not, it will most likely crack. In Torogh Dam the concrete temperature measurements have been done for various levels in different blocks (Figure 10). These measurements have been taken place in a ten year time period from 1994 to 2003. The temperature of concrete body in each block and each level has been measured in three layers. The upstream, middle and downstream layers the back analyses are described as follow.

The average temperature in concrete body In crest level has been 18°C in the upstream layer 21°C in the middle layer and 17°C in the down stream in winter season. As the measurements indicate, the temperature changes in the middle layer are negligible and the upstream and downstream layers depended on the temperature of surrounding environment.

In middle level, the more distance downward from the crest, the less the temperature of concrete body obtain. The temperature changes are not considerable and the highest temperature of the middle layer of concrete body has been occurred in winter, and the downstream layer depends on the environment temperature and the upstream layer relies on the water temperature.

In foundation level, the temperature changes of concrete body from the middle layer toward the foundation level showed an obvious decrease of temperature.

4.4 *Pendulum measurement*

Pendulum measurements and their locations in Torogh dam are shown in Figure 11.

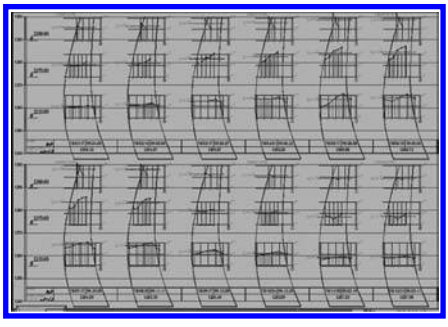

Figure 10. concrete temperature for various levels in Torogh dam.

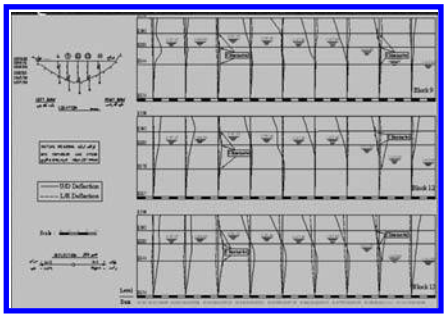

Figure 11. Tangible and radial displacement for various levels in Torogh dam.

Table 2. Pendulum measurement in Torogh dam.

Depth [m]	X [mm]	Y [mm]
0	214	146
-2,5	214	131
-5	198	139
-10	199	133
-15	196	141
-20	213	117
-25	220	113
...	...	...

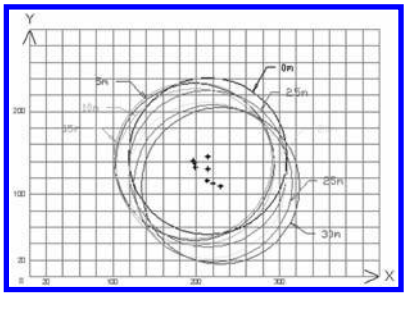

Figure 12. The carve for pendulum measurement.

These measurements which are related to a ten-year period (1994–2003). The tangible and radial deflections have been considered by direct pendulums. The most radial deformations have been occurred in middle block and at the crest level. The measurements will be presented in a Table 2. like the one block. The carve for pendulum measurement is shown in Figure 12.

5 CONCLUSIONS

The back analysis was done in 2 dams, Kardeh and Torgh. Both of them are double arch concrete dams, and constructed in Khorasan County and they are located in North east of Iran. The measurement period was in the range of 1994 to 2003 and the back analysis results are as follow.

In Kardeh Dam, the minimum concrete temperature occurred in the middle block and foundation level. The most radial deflection was in the crest level toward the lake and it took place at the minimum water level. The most tangible deflection occurred in the left abutment and at the middle water level. The unusual phenomena of drought caused the back analysis results differ from the calculated amounts but it lies in an allowable range.

In Torogh Dam, the Torogh Dam behavior was similar to Kardeh Dam since both dams are located in the same county (Khorasan) and they are only 150 kilometers far from each other. In this dam because of the body thickness the temperature phase delay of water and concrete was more than the Kardeh Dam.

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Impermeabilización y drenaje de la presa de Urdalur (Navarra)

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Confederación Hidrográfica del Ebro. Ministerio de Medio Ambiente y Medio Rural y Marino, Zaragoza, España

RESUMEN: La presa de Urdalur situada en el cauce del río Alzania en Navarra, es una presa de gravedad de hormigón compactado con rodillo (HCR). Su puesta en funcionamiento data de 1996 y desde el principio de su explotación, e incluso antes, ha manifestado importantes problemas de filtraciones. Tras una pequeña descripción de la presa y de su historia, se presenta en este artículo la evolución de las filtraciones en el tiempo. Se expone el análisis estadístico de las filtraciones realizado en el año 2009, que preveía una situación complicada a un año vista, lo que se ha visto confirmado, siendo precisa la declaración del Escenario 0 del Plan de Emergencia en 2010. Se describe también el plan de inyecciones y drenaje proyectado como consecuencia de lo anterior.

ABSTRACT: Urdalur dam, located on the river Alzania in Navarra, is a roller compacted concrete dam (RCC). Since the beginning of its operation, in 1996, and even before, significant filtration problems were observed. After a brief description of the dam and its history, this paper presents the evolution of the filtering over time. The statistical analysis of the leakage process accomplished in 2009, which provided a complicated situation one year ahead is also included. This pessimistic forecast has been confirmed, and in 2010 its been necessary to declare the level 0 of the emergency, according to the Dam Emergency Plan. The injection and drainage plan projected as a remedial measure is also described.

1 DESCRIPCIÓN DE LA PRESA

La presa de Urdalur se encuentra situada en el cauce del río Alzania, en Ciordia (Navarra) y conforma el embalse del mismo nombre que se sitúa sobre los términos municipales de Segura en Guipúzcoa y el ya citado de Ciordia en Navarra. El embalse tiene una capacidad de 5.4 hm^3.

Su función es la de regular las aportaciones de la cuenca del río Alzania para atender las demandas de abastecimiento urbano de unos 30,000 habitantes y de los polígonos industriales de la zona. En relación con esta funcionalidad, el embalse asegura, con una garantía del 90%, un caudal medio de 310 l/s.

En cuanto a la presa (en la figura 1 se presenta una vista desde agua abajo), se trata de una estructura de gravedad de planta recta y de hormigón compactado con rodillo (HCR). Su longitud total es de 396 m repartidos a lo largo de 6 bloques, numerados de 0 a 5 y separados por cinco juntas transversales con doble junta water-stop en el talud de aguas arriba.

En el bloque 3 (uno de los centrales) se sitúa el aliviadero, formado por tres vanos de labio fijo de 8.43 m de longitud cada uno y separados por pilas de 1 m de ancho.

El paramento de aguas arriba es vertical, mientras que el de aguas abajo es escalonado, con una inclinación media de 0.75H/1V.

La coronación se sitúa a la cota 652.00 sobre el nivel del mar y permite el paso de vehículos. Tiene 6 m de ancho.

La altura de la presa sobre cauce es de 47 m y de 55 m sobre cimientos.

Figura 1. Vista de la presa de Urdalur desde agua abajo.

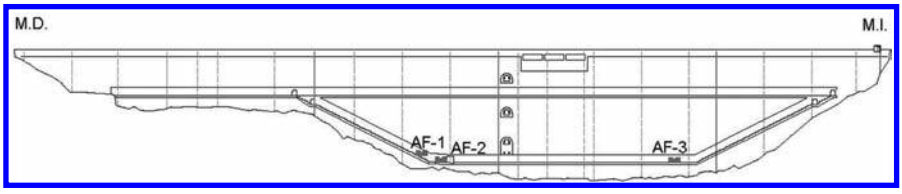

Figura 2. Alzado de la presa. Situación de las galerías y de los aforadores.

La estructura de coronación está formada por un puente que cruza el aliviadero mediante tres vanos de 8.43 m de luz cada uno. Las dos pilas y los dos estribos son los tajamares del aliviadero.

El ancho del cuerpo de presa en su cimentación es de aproximadamente 42 metros. Dispone de dos galerías de inspección, una perimetral inferior y otra superior, a media altura, que atraviesan la presa de estribo a estribo. Ambas tienen sección rectangular abovedada de 2 m de ancho por 2.5 m de alto (se reflejan estas galerías en la figura 2).

Verticales y paralelos al talud de aguas arriba, se encuentran situados los drenes para el control de las filtraciones del cuerpo de presa, realizados mediante perforaciones de 65 mm de diámetro y separados 3 m entre sí.

El desagüe de fondo se encuentra situado en la margen derecha de la presa, junto al estribo derecho del aliviadero. Está constituido por dos conductos de sección rectangular de 700 × 800 mm y de 5 m de longitud que emboquillan en el paramento de aguas arriba de la presa y se unen mediante una pieza de transición de 1,5 m a dos conductos metálicos de 800 mm de diámetro que discurren, completamente recubiertos de hormigón, por la parte inferior de la galería. Desembocan en el paramento de aguas abajo en la caseta de válvulas, donde dos válvulas de tipo Bureau de asiento plano realizan la regulación de caudales. Tras ellas se vierte el agua al cuenco amortiguador.

El caudal de abastecimiento entra en el sistema en tres tomas dobles de diámetro 500 mm, protegidas por rejillas para evitar la entrada de sólidos grandes, a las cotas 612.50, 625.50 y 640.50 m.s.n.m. respectivamente, que emboquillan en el paramento derecho aguas arriba, junto al estribo del aliviadero. Las tomas superior e intermedia descienden por el paramento de aguas abajo hasta juntarse todas en la cámara de válvulas.

2 GEOLOGÍA DE LA CERRADA

En la cerrada, el río Alzania discurre por un amplio valle en forma de artesa cuyo fondo es sensiblemente horizontal.

En la ladera izquierda, el espesor de suelos coluviales es del orden de un metro, existiendo numerosos afloramientos rocosos, predominantemente de limolita.

El fondo del valle está ocupado por una terraza aluvial sobre la que se han depositado algunas capas de suelos de origen coluvial, derrubios de ladera o suelos depositados por el río con un arrastre muy corto. El espesor medio de suelos en esta zona puede estimarse en 4 m.

Entre el cauce del río Alzania, que está totalmente excavado en la roca, y el pie de la ladera derecha, los suelos naturales quedan cubiertos por un relleno artificial de unos 4 m de espesor máximo, realizado para dar paso a un camino forestal.

3 BREVE HISTORIA DE LA PRESA

La presa de Urdalur ha pasado por un gran número de soluciones técnicas, desde el primer anteproyecto redactado en el año 1960 hasta el inicio de las obras, y su posterior puesta en funcionamiento en el año 1996.

En 1993, durante la construcción de la presa, se produjeron desprendimientos importantes de los taludes situados junto al estribo derecho, por lo que se hizo necesario rellenar los huecos existentes entre la presa y los taludes de aguas arriba y aguas abajo, previa colocación de un dren y con una protección de escollera como terminación.

En 1995, poco antes del inicio de la explotación, aparecieron filtraciones importantes a través de los drenes de la galería inferior y algunas juntas de pantalla. Esto dio lugar a la ejecución de una campaña de inyecciones.

Para la ejecución de estas inyecciones se aprovecharon las perforaciones de drenaje existentes por lo que, al quedar inutilizada la pantalla, fue necesario realizar una nueva, ejecutada en vertical, desde la galería perimetral y adosada al hastial aguas abajo de la galería. En las perforaciones se instalaron llaves de tres vías, para el control de filtraciones y presiones, en las cabezas de drenes.

En noviembre de 1996 comenzó la explotación del embalse, haciéndose cargo de ella la Confederación Hidrográfica del Ebro.

Ya en el comienzo de la explotación, en 1997, se produjo un fallo en la soldadura longitudinal

del conducto izquierdo de los desagües de fondo, dentro de la galería, siendo necesaria su reparación.

Poco después, el 5 de junio de 1999 se detectó una fuga de gran caudal, del orden de 2 m^3/s, como consecuencia de la rotura de la pieza de transición entre la tubería del desagüe de fondo y la compuerta Bureau, que inundó la galería y afloró a través de la cámara de válvulas de aguas abajo. Se decidió, entonces, realizar la rehabilitación total del desagüe de fondo. Para hacer posible la obra sin el vaciado total del embalse, se colocó un escudo adosado al paramento de aguas arriba y se sustituyeron los dos desagües de fondo (que se recubrieron de hormigón) y las dos tomas de agua inferiores. En este proceso quedó inutilizado el sistema de medida de filtraciones usado hasta entonces, lo que dio lugar a una discontinuidad en la serie de datos registrados.

Por otra parte y debido a las filtraciones en el pozo en el que se aloja el péndulo inverso, los depósitos de carbonatos acumulados en los instrumentos de lectura, acabaron por inutilizar el sistema.

Como consecuencia de los incidentes se decidió abordar la regeneración del sistema de auscultación. Así, en mayo de 2006 entró en funcionamiento el nuevo sistema de auscultación que incluye, obviamente, el control de filtraciones.

En agosto de 2008 se realizó un ajuste de las regletas de los aforadores y en julio de 2009 se comienzan a recoger los datos de los piezómetros de cuerda vibrante.

4 DESCRIPCIÓN DEL ACTUAL SISTEMA DE AUSCULTACIÓN

Se dispone de 6 piezómetros de cuerda vibrante para el control de las presiones intersticiales en el cimiento de la presa en estado operativo. Otros dos, instalados simultáneamente, han fallado y prácticamente desde su instalación proporcionan datos erróneos.

Los caudales de filtración se controlan en tres aforadores triangulares tipo Thomson que están instalados en las galerías de la presa. De ellos, dos, AF-1 y AF-2 son de lectura automática y el otro AF-3 sólo de lectura manual (ver figura 2).

5 LA PROBLEMÁTICA ACTUAL

Desde que se reinició la lectura de la instrumentación (primavera de 2.006) se viene observando un crecimiento constante en los valores de los caudales aforados, lo que ha motivado su seguimiento en detalle.

Dentro de este seguimiento, al finalizar el año 2.008 se ha efectuado un análisis de la situación, que es la que se expone en los puntos que siguen.

6 LA SITUACIÓN AL FINAL DE 2008

En relación con las filtraciones, el análisis realizado permite establecer que han existido tres periodos desde el inicio de la explotación hasta nuestros días. Estas etapas se tratan en los puntos que siguen.

6.1 *Etapa inicial*

Iría desde noviembre de 1996 a junio de 1999, cuando se produjo la inutilización del sistema de medida por la rotura del desagüe de fondo.

En este periodo se observa que la evolución de las filtraciones está relacionada con el nivel de llenado del embalse, como queda claramente expresado en la figura 3, en la que se refleja la evolución temporal del nivel de embalse y del caudal total aforado. No se detecta ningún síntoma de anomalía.

Estas medidas fueron las utilizadas para la elaboración del Plan de Emergencia y la determinación de umbrales en relación con las filtraciones.

6.2 *Etapa intermedia*

Se asigna este periodo al comprendido desde junio de 1999 hasta mayo de 2006, cuando entra en funcionamiento el nuevo sistema de auscultación.

En este periodo la toma de datos fue realizada manualmente, prestando especial importancia a la filtración que se localizaba en la junta entre los bloques 2 y 3.

En la figura 4 se refleja la evolución temporal de las filtraciones en relación con la del nivel de embalse. Se aprecia cómo se produce un incremento muy importante en las filtraciones durante la primavera de 2003 cuando el volumen del embalse venía manteniéndose prácticamente constante.

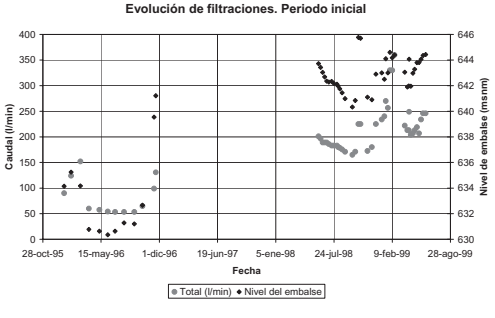

Figura 3. Evolución temporal de las filtraciones en la primera etapa.

Como respuesta a esto se ejecutó una campaña de inyecciones, utilizando resinas y microcemento. El resultado aparente fue bueno, consiguiéndose sellar la filtración de la junta entre los bloque 2 y 3, pero a partir de octubre de 2005 vuelve a surgir una pequeña filtración en la citada junta, que parece evoluciona con el nivel de embalse.

6.3 *Etapa tercera*

Corresponde al periodo comprendido entre el momento en que entra en funcionamiento el nuevo sistema de auscultación hasta el final de 2008, momento en el que se realiza el análisis.

En la figura 5 se representa la evolución temporal del nivel de embalse conjuntamente con la de los caudales aforados. Se incorpora en el gráfico también el valor que marca el umbral del escenario 0 de emergencia (función del nivel de embalse, de acuerdo con le Plan elaborado).

Inicialmente es preciso señalar que aparece una diferencia apreciable (de 5.8 l/s a 3.8 l/s) en la filtración total y entre los valores final de la figura 4 e inicial de la figura 5, si bien tan solo es un salto aparente, ya que está motivado por el cambio de sistema de medida.

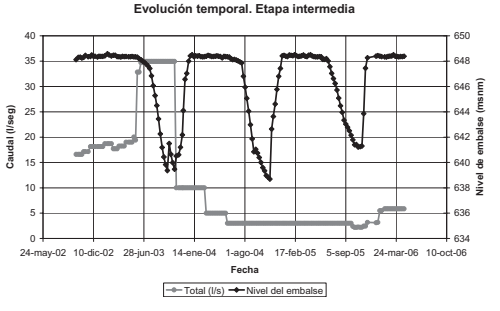

Figura 4. Evolución temporal de las filtraciones en la segunda etapa.

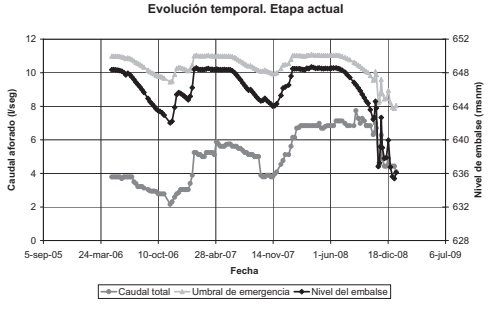

Figura 5. Evolución temporal de las filtraciones en la tercera etapa.

Se aprecia una tendencia claramente marcada y creciente de las filtraciones a lo largo de todo el periodo, así como la aproximación que se estaba produciendo hacia el umbral del escenario 0 de emergencia.

6.4 *Análisis estadístico*

Con el fin de intentar expresar los datos conocidos en una forma más sintética y que permita en mayor medida su análisis, se ha realizado un análisis estadístico de un cierto nivel de detalle de las medidas de las filtraciones en el último periodo (a partir de junio de 2006).

En primer lugar se ha planteado la posible relación de las medidas con el nivel de embalse y la época del año (efectos reversibles) y con el tiempo (efectos irreversibles).

Se ha considerado la influencia del nivel de embalse considerando una relación de hasta potencia 4. La influencia de la época del año (que sustituye como medida indirecta a la influencia de la temperatura) se ha introducido mediante funciones trigonométricas (seno y coseno, llegando hasta la potencia 2) de un ángulo que recorre los 360° a lo largo del año. Por su parte, el efecto del tiempo se ha evaluado por composición de una relación lineal con relaciones exponenciales directa e inversa (el año corresponde a una unidad).

La relación finalmente obtenida es la siguiente:

$$Q = 2.689 - 1.645\cdot t + 0.452\cdot e^{t} - 4.264\cdot e^{-t} + 0.204\cdot sen(\theta) - 0.435\cdot sen(\theta)\cdot\cos(\theta) + 3.820\cdot H + 1.049\cdot H^{3} \quad (1)$$

En ella se utiliza la siguiente notación:

- Q = Caudal total de filtración (l/s)
- t = Tiempo expresado en años y medido desde el 1-5-06
- θ = ángulo que expresa la época del año, correspondiendo los valores 0 y 360° al día 1-5-06
- H = cota relativa del nivel de embalse, correspondiendo el valor 0 a la cota 635,38 y el valor 1 a la 648,69 (mínimo y máximo registrado en el intervalo de medidas)

El ajuste obtenido es bueno, como se pone de manifiesto de forma gráfica en la figura 6, en la que se refleja el valor estimado contra el valor realmente medido, junto con los límites de confianza del 95% de la estimación.

Expresando lo anterior de una forma más numérica, el ajuste presenta las siguientes características:

Los parámetros ajustados son:

Por otra parte, en la figura 7 se reflejan de manera gráfica los valores estandarizados de los coeficientes de estimación junto con el rango de

Tabla 1. Resultados del ajuste del modelo completo.

N° de observaciones		139,000
Grados de libertad		131,000
R^2		0,965
R^2 ajustado		0,964
Error cuadrático medio		0,078
Varianza	Explicada	286,485
	Remanente	10,275
	Inicial	296,761

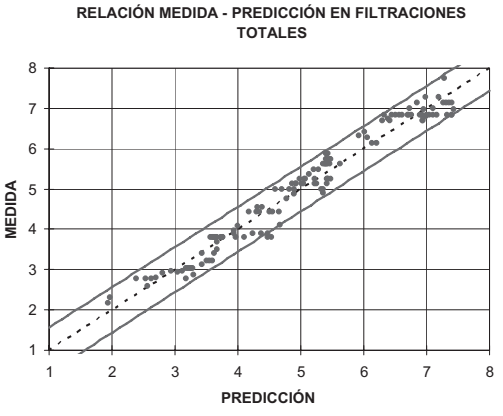

Figura 6. Filtración estimada frente a filtración total medida. Modelo completo.

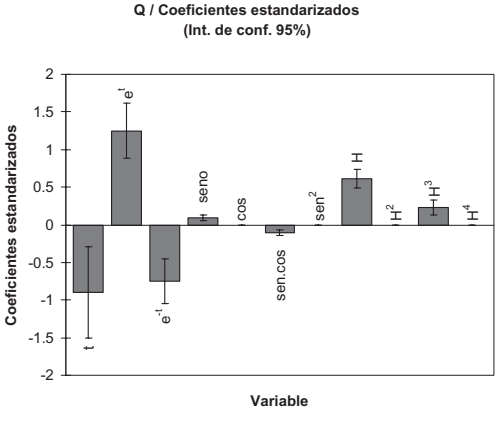

Figura 7. Coeficientes de ajuste estandarizados y variabilidad al 95%. Modelo completo.

variación que corresponde al intervalo de confianza del 95%.

En esta figura se aprecia la poca incidencia que tienen en el modelo las variables explicativas relacionadas con la época del año (las expresadas como funciones trigonométricas de θ). Por otra parte, el orden de entrada en el modelo se recoge en la Tabla 3 (en ella, MEC es el error cuadrático medio).

Tabla 2. Parámetros del ajuste del modelo completo.

Fuente	Valor	Desviación típica
Intersección	2,689	0,643
t	–1,645	0,557
e^t	0,452	0,067
e^{-t}	–4,264	0,858
$Sen(\theta)$	0,204	0,039
$Cos(\theta)$	0,000	0,000
$Sen(\theta)\cdot Cos(\theta)$	–0,435	0,072
$sen^2(\theta)$	0,000	0,000
H	3,820	0,402
H^2	0,000	0,000
H^3	1,049	0,227
H^4	0,000	0,000

En ella se aprecia cómo el modelo puede considerarse razonable sin tener en cuenta el factor asociado a la época del año. En relación con la incidencia de la evolución temporal, parece inevitable la consideración del factor lineal (variable t), manteniéndose la duda de la incidencia real del factor exponencial positivo (e^t): Su consideración hace que el error cuadrático medio se reduzca de 0.153 a 0.128 y que el coeficiente de regresión ajustado suba desde 0.929 a 0.941.

Esto ha conducido a plantear estos dos modelos como alternativos: modelos basados en t, H y H^3 y en las mismas variables más e^t.

Las ecuaciones que se obtienen son las siguientes:

$$Q = -0.345 + 1.698\cdot t + 3.116\cdot H + 0.971\cdot H^3 \quad (2)$$

$$Q = -0.940 + 0.994\cdot t + 4.312\cdot H + 0.530\cdot H^3 + 0.165\cdot e^t \quad (3)$$

La potencia de ambos modelos como explicativos de la realidad observada se refleja en las figura 8 (modelo reducido sin considerar la exponencial del tiempo como variable explicativa) y la figura 9 (modelo reducido incluyendo la exponencial del tiempo entre las variables explicativas). En ellas se reflejan las predicciones que proporciona el modelo frente a las medidas observadas, apreciándose la al menos aparente validez de ambos modelos.

Como complemento a las figuras anteriores, en la figura 10 se refleja la comparación entre las predicciones de ambos modelos. Se aprecia la muy ligera diferencia entre ambas predicciones.

Numéricamente, en las tablas 4 y 5 se reflejan las características de los ajustes realizados y los parámetros obtenidos con ambos modelos.

Del mismo modo, en las figura 11 y 12 se reflejan los coeficientes estandarizados junto con sus intervalos de confianza del 95%.

Tabla 3. Proceso de conformación del modelo completo.

Variables	Variable (In/Out)	MEC	R^2	R^2 ajustado
e^{-t}	e^{-t} (In)	1,072	0,505	0,501
e^{-t} / H^3	H^3 (In)	0,384	0,824	0,821
t / e^{-t} / H^3	t (In)	0,223	0,899	0,897
t / H^3	e^{-t} (Out)	0,221	0,899	0,897
t / H / H^3	H (In)	0,153	0,930	0,929
t / e^t / H / H^3	e^t (In)	0,128	0,942	0,941
t / e^t / *sen.cos* / H / H^3	*sen.cos* (In)	0,104	0,953	0,952
t / e^t / *sen* / *sen.cos* / H / H^3	*sen* (In)	0,093	0,959	0,957
t / e^t / e^{-t} / *sen* / *sen.cos* / H / H^3	e^{-t} (In)	0,078	0,965	0,964

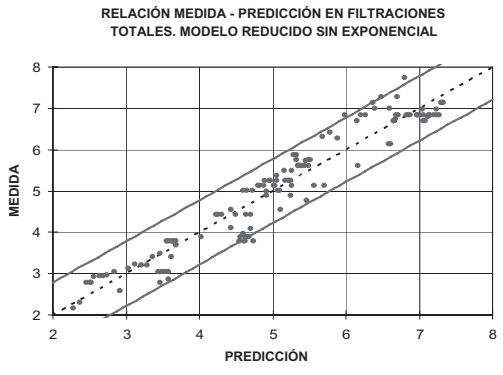

Figura 8. Filtración estimada frente a filtración total medida. Modelo reducido sin e^t.

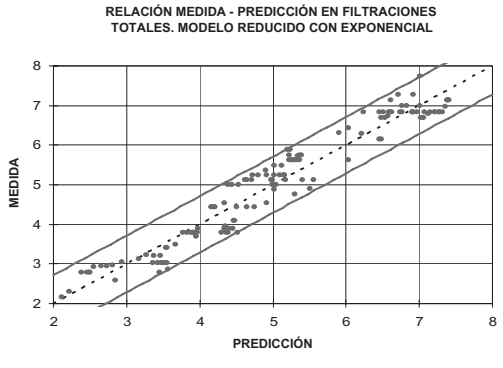

Figura 9. Filtración estimada frente a filtración total medida. Modelo reducido con e^t.

La observación de estas últimas figuras pone de relieve el importante cambio que induce la consideración del factor e^t, llegándose hasta el extremo de modificar no solo la incidencia de t sino también de H^3 (podría incluso llegar a desaparecer, al aparecer el 0 casi en el interior del intervalo de confianza del 95%).

De cara a intentar precisar un poco más la situación se ha procedido a evaluar los residuos

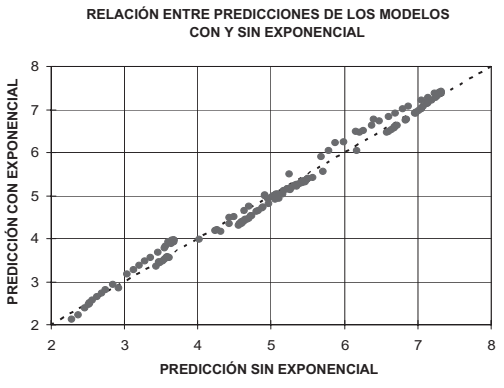

Figura 10. Relación entre predicciones de los modelos reducidos con y sin consideración de e^t.

Tabla 4. Características de los ajustes. Modelos reducidos.

Modelo		Sin e^t	Con e^t
N° de observaciones		139,000	139,000
Grados de libertad		135,000	134,000
R^2		0,930	0,942
R^2 ajustado		0,929	0,941
Error cuadrático medio		0,153	0,128
Varianza	Explicada	276,040	279,617
	Remanente	20,720	17,144
	Inicial	296,761	296,761

Tabla 5. Parámetros de los modelos reducidos.

	Modelo sin e^t		Modelo con e^t	
Fuente	Valor	Desviación típica	Valor	Desviación típica
Intersección	–0,345	0,213	–0,940	0,225
t	1,698	0,047	0,994	0,140
H	3,116	0,400	4,312	0,429
H^3	0,971	0,276	0,530	0,266
e^t			0,165	0,031

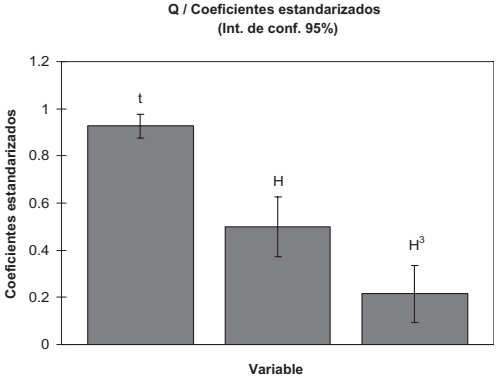

Figura 11. Coeficientes estandarizados. Modelo reducido sin e^t.

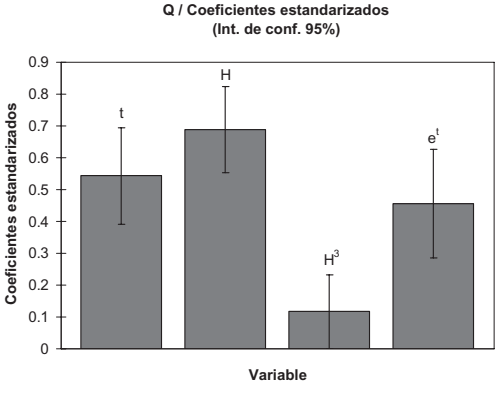

Figura 12. Coeficientes estandarizados. Modelo reducido con e^t.

resultantes de la consideración del modelo reducido sin incluir e^t en relación con la posibilidad de incluir esta última variable. En la figura 13 se refleja la evolución temporal de estos residuos (l/seg), apreciándose ya la no apariencia de correlación clara. Aplicando criterios estadísticos, el coeficiente de correlación de Pearson que se obtiene entre los residuos y la variable e^t es de tan solo 0.10, lo que lleva a, en principio y a falta de otras evidencias, a rechazar la inclusión de la exponencial del tiempo como variable explicativa de la evolución de las filtraciones.

A la vista de todo lo expuesto puede concluirse que la evolución temporal de las filtraciones registradas en la presa y en el periodo analizado es razonablemente bien explicada mediante un modelo en el que se incluyen como variables explicativas las relacionadas con el tiempo y con el nivel de embalse.

El nivel de embalse se introduce mediante la consideración de las potencias 1 y 3, con gran

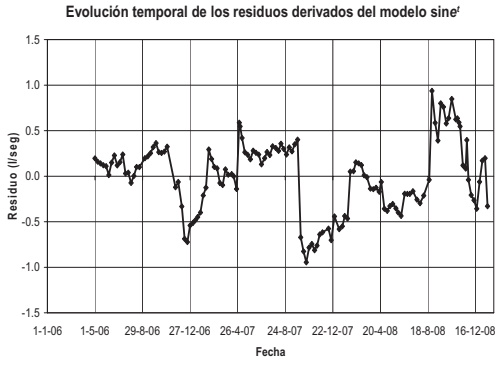

Figura 13. Evolución temporal de los residuos después de aplicar el modelo reducido sin e^t.

preponderancia de la primera. Esto puede ser interpretado (con las incertidumbres que se derivan necesariamente de la consideración de tan solo análisis estadísticos sobre medidas, sin profundizar en el tema mediante inspección o pruebas, por ejemplo) como que la mayor parte de la filtración registrada se relaciona fundamentalmente con el gradiente y en mucha menor medida con la superficie filtrante. Los datos estadísticos pueden estar poniendo de relieve la existencia de una zona de filtración preferente en las zonas bajas de la presa (en zonas por debajo del nivel mínimo de embalse registrado en este periodo), de forma que la filtración aportada por esta zona se incrementa en razón directa al incremento de carga (de nivel) del embalse y se complementa con un cierto aporte de otras zonas que crecen superficialmente (con la potencia 2) al subir el nivel de embalse a la vez que se incrementa el gradiente (linealmente).

La influencia del tiempo es clara, con una dependencia lineal y siendo más dudosa la influencia de e^t. Esta disquisición tiene una cierta trascendencia ya que implica discriminar entre una evolución constante y otra progresiva. El análisis realizado conduce a considerar como más probable la hipótesis de no progresividad, lo que conduce a pensar que, caso de existir esta, no será muy marcada, al menos en el presente.

Desde un punto de vista más numérico, podía esperarse que una variación en el nivel de agua de unos 13 m (oscilación máxima en el periodo considerado) diese lugar a una modificación del caudal de aproximadamente 4.1 l/seg, de los cuales 3.1 seguirían una evolución lineal y el resto seguiría una evolución cúbica. También era esperable que cada año se produjese un incremento, independiente de las posibles modificaciones inducidas por cambios en el nivel de embalse, de aproximadamente 1.7 l/seg, si se mantuviese la evolución que estaban llevando entonces las filtraciones.

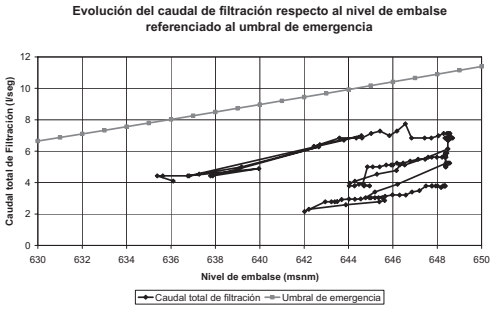

Figura 14. Evolución del caudal en relación con el nivel de embalse y en relación con el umbral de emergencia.

Desde el punto de vista de los umbrales de emergencia todavía se mantenía entonces un cierto margen respecto a ellos, como se pone de manifiesto en la figura 14.

En ella se aprecia que sobre una evolución muy lineal con el nivel de embalse se producía un crecimiento año a año. A fecha de mediados de enero de 2009, se mantenía un margen en relación con el umbral de emergencia que permitía una cierta tranquilidad durante aproximadamente un año si se mantuvieran las evoluciones que entonces aparecían.

En función de todo lo anterior, parecía necesario abordar algún tipo de actuación que permitiese reducir las filtraciones, para lo que se disponía de un cierto margen de tiempo, pero que, igualmente, tenía una cierta urgencia. Era evidente que una situación de embalse bajo, como era la que se estaba produciendo, no pudiera considerarse como situación de riesgo, pero también era claro que, previsiblemente, en un año iba a ser prácticamente imperativo el mantenimiento del embalse en un nivel bajo.

Para establecer la actuación a desarrollar era, sin duda, preciso un análisis de la situación y la elaboración del adecuado proyecto.

7 PROYECTO DE IMPERMEABILIZACIÓN

Como consecuencia de lo expuesto se ha elaborado un proyecto de rehabilitación en el que se plantea como objetivo el conseguir la impermeabilización del cuerpo de presa y la consolidación e impermeabilización de estribos de la presa de Urdalur.

Aunque, dado el historial de la presa, no se descarta que sean necesarias nuevas campañas de impermeabilización en un futuro más o menos próximo (lo que exige el mantenimiento del sistema de inspección y auscultación), el proyecto elaborado prevé la ejecución de las actuaciones que se exponen en los puntos que siguen.

7.1 *Pantalla de impermeabilización del cuerpo de presa*

Ejecución de una pantalla de impermeabilización en el cuerpo de presa, desde la coronación de la misma y en toda su longitud (400 m). Será ejecutada en dos fases, de manera que inicialmente se perforarán taladros separados 3 m entre sí y, en una segunda fase, completando la pantalla con un taladro entre cada dos existentes, con lo que se consigue una perforación cada 1.5 m. Se prevé penetrar un metro por debajo del contacto presa-terreno y la utilización como elemento de inyección el cemento y la lechada de microcemento.

7.2 *Tratamiento de consolidación*

Como complemento a la pantalla de impermeabilización del cuerpo de presa, se realizarán dos pantallas de consolidación desde la galería perimetral partiendo de cada uno de los estribos y una longitud de 153 m y 50 m respectivamente (estribo derecho y estribo izquierdo). En los extremos de la galería los taladros tendrán una inclinación de 5° aguas arriba y aguas abajo y una longitud de 10 m. Además en el estribo derecho se ejecutarán 9 taladros inclinados de 25 m para realizar el tratamiento de consolidación lo más completo posible. En el centro de la galería serán de 25 m en vertical.

Todos los taladros se inyectarán empleando lechadas de cemento.

7.3 *Reperforación de los taladros de drenaje*

Se realizarán nuevos taladros de drenaje en sustitución de los afectados por la realización de las pantallas de impermeabilización y consolidación.

Los nuevos taladros se harán entre los existentes y después de inyectados éstos y, estarán separados entre sí tres metros aproximadamente.

Los del cuerpo de presa tendrán una profundidad media de unos 44 m hasta la galería inferior, y los de subpresión de 15 m en el terreno desde la galería perimetral, inclinados 5° hacia aguas abajo.

Alrededor de los drenes que actualmente aportan mayores filtraciones, las inyecciones y el sellado de las fisuras se harán, en lugar de con cemento y microcemento, mediante resinas epoxi, con lo que se conseguirá una mayor eficiencia.

7.4 *Mediciones aproximadas*

Se prevé que se perforarán aproximadamente unos 19,000 m considerando la reperforación. La longitud anterior equivale a unos 700 taladros, inyectándose aproximadamente 710 toneladas de cemento, 14 toneladas de bentonita, 178 toneladas de microcemento A12 y 1000 litros de resinas expoxi.

7.5 *Presupuesto de las obras*

A partir de las mediciones de las unidades correspondientes cuyas unidades más importantes han quedado resumidas en el apartado anterior, se obtiene el Presupuesto de Ejecución Material(PEM) de las obras cercano a 1.5 millones de euros, lo que supone un Presupuesto Base de Licitación (PEM + 17% de Gastos Generales + 6% de Beneficio Industrial + 16% de IVA) ligeramente superior a los 2 millones de euros.

8 ETAPA ACTUAL

Como ya se ha puesto de manifiesto, el análisis de la situación presentado y la solución proyectada corresponden al final de 2.008. Con posterioridad a ese momento se dispone de más información, que es la que se presenta aquí.

Así, en la figura 15 se refleja la evolución en el último año de las filtraciones a través del cuerpo de presa en relación con la evolución del nivel de embalse.

En ella se aprecia cómo a principios del mes de noviembre de 2009 las filtraciones aumentan de forma brusca, coincidiendo con el aumento del nivel del embalse, a su vez consecuencia de las importantes precipitaciones registradas. Se pasa de la cota 628 a la 643.

A partir de este mes el nivel del embalse se mantiene estable, alrededor de la cota 643 m.s.n.m., pero las filtraciones totales de la presa aumentan de forma paulatina, desde 320 l/min hasta alcanzar un máximo de 555 l/min los días 10 y 11 de enero de 2010. Posteriormente se reduce el nivel del embalse, descendiendo las filtraciones hasta los 394 l/min registrados el 20/1/2010.

Como complemento a lo anterior, en la figura 16 se reflejan las filtraciones en el cuerpo de presa en función del nivel de embalse y en relación con el umbral del escenario 0 de emergencia. Se aprecia cómo ya se ha superado el umbral establecido, lo que se ha producido en el inicio de diciembre de 2.009 (concordante con la previsión realizada un año antes).

Adicionalmente a lo anterior, en ese momento (realmente, a partir de julio de 2.009) se dispone ya de medidas de los piezómetros de cuerda vibrante instalados. Como ilustración, en la figura 17 se reflejan las medidas del PCV-1, situado en el bloque 2. Los resultados indican una elevada presión, que llega a ser del orden del 50–60% de la carga hidrostática a unos diez metros de profundidad del contacto entre presa y cimiento; esto parece indicar la existencia de una vía de agua a través de la cimentación de la presa.

Las consideraciones anteriores junto a la constatación, consecuencia de la inspección, de la existencia de depósitos de material térreo en los drenes de cimentación (síntoma de algún tipo de pérdida de material del cimiento) y de una surgencia en la margen derecha que no está aforada, pero que podría tratarse de agua que pasa a través del cimiento o del estribo derecho han motivado que el día 25 de enero de 2.010 se declarase formalmente el Escenario 0 del Plan de Emergencia (lo que implica esencialmente el incremento de la observación de la presa), estableciéndose una limitación al llenado del 50% de la capacidad del embalse como medida de cautela, en tanto en cuanto no se ejecuten medidas correctoras proyectadas.

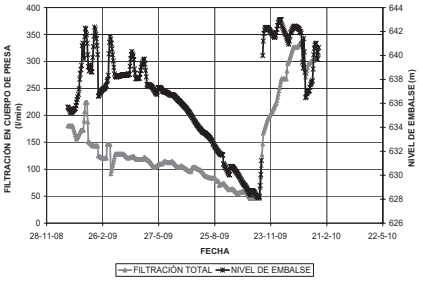

Figura 15. Evolución de las filtraciones a través del cuerpo de presa en relación con el nivel de embalse durante el último año.

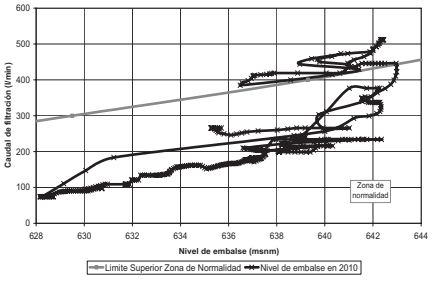

Figura 16. Evolución de las filtraciones a través del cuerpo de presa en relación con el nivel de embalse respecto al umbral del escenario 0 de emergencia.

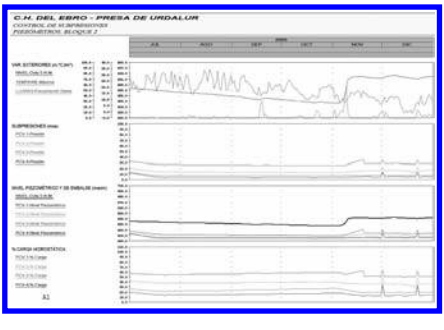

Figura 17. Evolución de las medidas del PCV-1, situado en el bloque 2.

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Deterioration processes of a land dam in Mexico

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ABSTRACT: The present paper focuses on research possible causes of deterioration processes the curtain of land dam "El Batan" in Querétaro, Mexico, including the design of the fineness of the employee filter during the construction of the curtain was verified since this depends greatly on the proper functioning of this filter. To carry out the required analysis, it was necessary to document elements provided understanding about the composition and behavior of the land curtain, and the main types of failure in these curtains.

The general characteristics of the land dam curtain "El Batan", the composition of the filter, as well as possible causes resulted in the failure were also analyzed. Once obtained data starting, the actual analysis was carried out by reviewing the following possible deterioration processes: fails due to a poor constructive process of the curtain, failure due to hydraulic suppression, fails due to a structural design wrong, fails due to a geotechnical design wrong, fails due to a hydraulic design wrong, fails due to an inadequate design of the curtain filter.

It is concluded that the type of the filter employed in the land dam curtain of "El Batan", located in the municipality of Querétaro, México, do not have adequate characteristics, outside of the ranges of design, using the curves: Terzaghi criteria, Sherard and Dunnigan criteria, UCSCS criteria, and Foster and Fell criteria.

1 INTRODUCCIÓN

En los ríos del país de México escurren aproximadamente 400 kilómetros cúbicos de agua anualmente, incluyendo las aguas que entran desde los países vecinos y descontando los que salen hacia ellos. Aproximadamente 87% de este escurrimiento ocurre en 39 ríos principales, cuyas cuencas ocupan cerca del 58% de la extensión territorial continental.

Los ríos que acaparan el 65% del escurrimiento son el Grijalva-Usumacinta, Papaloapan, Coatzacoalcos, Balsas, Pánuco, Santiago y Tonalá (cuyas cuencas en conjunto totalizan el 22% del territorio nacional).

Los ríos Balsas y Santiago pertenecen a la vertiente del Pacífico y los otros cinco a la vertiente del golfo de México. Por su longitud destacan los ríos Bravo y Grijalva-Usumacinta.

Figura 1. Principales ríos de México.

1.1 *Las presas de México*

Una parte del agua que escurre por los ríos es actualmente almacenada en presas y se utiliza, entre otros fines, para la producción de alimentos, la generación de energía eléctrica, el control de las inundaciones y el suministro de agua para los usos industrial y doméstico.

Las principales presas del país de México comenzaron a construirse antes de 1920 y tuvieron un crecimiento acelerado entre 1940 y 1970. De las 4000 presas existentes, 667 están clasificadas como grandes presas de acuerdo con los criterios de la Comisión Internacional de Grandes Presas. La capacidad de almacenamiento proporcionada es

Figura 2. Principales presas de México.

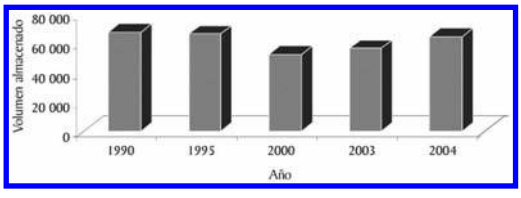

Figura 3. Volumen almacenado en las principales presas de México, en Hm^3.

de 150 kilómetros cúbicos de agua y en conjunto equivaldría al 37% del escurrimiento promedio anual del país. Sin embargo, el promedio del volumen de almacenamiento registrado en las 51 presas principales del país entre 1990 y 2004 fue de 61 kilómetros cúbicos.

Las presas que registraron los mayores volúmenes promedio de almacenamiento (considerando los años 1990, 1995, 2000, 2003 y 2004) fueron La Angostura y Malpaso (ambas en el estado de Chiapas, con 10,500 y 8,500 hectómetros cúbicos por año, respectivamente), Infiernillo (Michoacán, 7,600 hectómetros cúbicos) y Temascal (Oaxaca, con cerca de 5,000 hectómetros cúbicos).

2 DESCRIPCION DE LA PRESA DE TIERRA "EL BATAN"

La presa "El Batán" está ubicada sobre el río Pueblito, afluente del río Laja, tributario a su vez del río Lerma, a unos 5.5 km aguas arriba de la localidad Pueblito, del municipio de Corregidora del estado de Querétaro, cuyas coordenadas 20°30'13" de latitud norte y 100°24'29" de longitud oeste.

Esta presa se destina a aprovechar el escurrimiento del río Pueblito para riego de una superficie de 187 Ha de terrenos ribereños y para dotar de agua potable a 25,000 habitantes del municipio de Corregidora. La presa está formada por una cortina de materiales graduados de 207.5 m de longitud por la corona y 45.5 m de altura

Figura 4. Localización geográfica de la presa "El Batán".

Figura 5. Vista aérea de la presa "El Batán".

máxima, que forma un embalse con capacidad de 8.77 millones de m^3; una obra de toma del tipo de torre de toma y tubería de presión, ubicada en la ladera izquierda, con capacidad de diseño para un gasto de 1,200 l/s y una obra de excedencias consistente en un vertedor del tipo de cresta con canal lateral, situado en el mismo lado, con capacidad de diseño de 534 m^3/s.

2.1 *Cortina de la presa*

La cortina de la presa "El Batán" es de materiales graduados y tiene 207.5 m de longitud por la corona, que es de 10.0 m de ancho, se encuentra a la elevación de 1,907.15 msnm y presenta una altura máxima de 45.5 m. La sección transversal está constituida por un corazón impermeable de arcilla compactada de 6.0 m de ancho en la parte superior con taludes de 0.4:1 en ambos lados, cubierto con filtros de grava-arena de espesor variable de 1.0 m en la parte superior a 2.0 m en el desplante, seguido

Figura 6. Cortina vista aguas arriba.

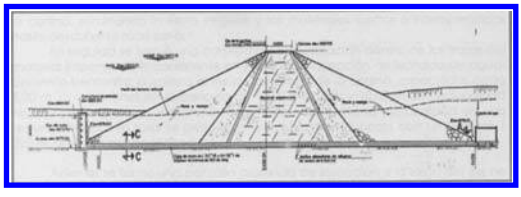

Figura 7. Sección transversal de la cortina de la presa.

de zonas de transición con taludes de 0.75:1, formadas con material de rezaga no mayor de 30 cm, confinadas mediante zonas de roca y rezaga con taludes 2:1 en los dos paramentos.

La corona está cubierta con una capa de tepetate compactado de 0.3 m de espesor, protegida con una capa de tezontle de 0.2 m con bombeo de 2% hacia ambos lados. A lo largo de la corona se colocaron fantasmas formados con tubos de asbesto-cemento de 15.2 cm (6") de diámetro rellenos de concreto reforzado y estribos de alambrón de 0.64 cm (1/4") de diámetro colocados cada 8.0 m a lo largo de ambos lados de la corona, que dejan una faja de circulación de 6.0 m de ancho.

3 PROCESO DE DETERIORO DE LA PRESA "EL BATAN"

3.1 *Comportamiento inicial*

Durante el verano del año de 1991, en el mes de julio, el embalse de la presa se llenó por primera vez en el lapso de tres semanas e inclusive derramó por el vertedor de excedencias una lámina hasta de 0.5 m, permaneciendo la descarga durante una semana aproximadamente. Los asentamientos registrados durante el primer llenado de la presa se consideraron normales.

En forma súbita, el día 28 de julio de 1991, ocurrió un accidente en la cortina que consistió en dos hundimientos con forma de cono invertido, localizados en los hombros de aguas arriba y aguas debajo de la corona, en una línea sensiblemente paralela al tajo de desvío, ligeramente desplazada hacia la ladera izquierda.

Asimismo se apreció aguas debajo de la cortina y en la zona de tajo del desvío, donde fue alojada la obra de toma, se presentó una filtración importante, que gradualmente aumentaba su caudal, considerándose que llegó a ser superior a 50 l/s.

Simultáneamente a la formación de los hundimientos de cono invertido, las filtraciones fueron disminuyendo hasta unos 15 l/s. Se apreció en el agua de la filtración el arrastre de partículas finas, de tonalidad negra, similar a la arcilla con la que se construyó el núcleo impermeable. Por lo anterior, se consideró que el fenómeno ocurrido en la cortina corresponde a un efecto de erosión interna en el núcleo impermeable.

Se procedió de inmediato a rellenar los conos, utilizando el mismo material empleado en la construcción de los respaldos, se abrió una escotadura en el cimacio del vertedor de excedencias y se llevaron a cabo las acciones necesarias para poder extraer por la toma el mayor caudal posible, con el objeto de reducir la carga hidrostática y disminuir el riesgo de una falla de la cortina por tubificación.

3.2 *Investigación de las causas de los hundimientos*

Se procedió a instalar vertedores de aforo para llevar un registro del caudal de las filtraciones y se inició la toma de muestras de los arrastres que tenían lugar en la zona de tajo de desvío. Las gráficas de variaciones del caudal de las filtraciones en relación con el nivel del embalse mostraron una buena correlación. Los resultados de los análisis efectuados en las muestras de los arrastres indicaron que su granulometría y características de plasticidad, coincidían con la del material empleado en la construcción, por lo que se consideró que el arrastre procedía del núcleo impermeable.

Con objeto de corroborar lo anterior, se procedió a investigar el núcleo impermeable con pozos a cielo abierto por el centro de la corona. El primer pozo se excavó en el cadenamiento 0 + 055 hasta una profundidad de 18 m, fue posible apreciar pequeñas grietas sensiblemente horizontales con sus planos inclinados hacia aguas abajo y hacia aguas arriba, que dejaban superficies brillantes como si hubiese habido un deslizamiento. Un segundo pozo de 20 m de profundidad se excavó en el cadenamiento 0 + 063 observándose los mismos fenómenos que en el otro pozo.

Para continuar la investigación hasta la roca de desplante, se perforó en el fondo del pozo un barreno de 6" de diámetro, brotando agua a una profundidad de 6.0 m, alcanzando un nivel de 2.0 m inferior al nivel del embalse.

Para tener mayores elementos que permitieran definir las causas del fenómeno ocurrido, se

consideró conveniente instalar dentro de la zona del tajo de desvío una serie de piezómetros localizados tanto aguas arriba como aguas abajo, algunos alojados dentro del núcleo impermeable y otros en la roca de cimentación.

3.3 *Resultados*

Como resultado de esta investigación de los hundimientos presentados, se encontró que durante la perforación de algunos barrenos para alojar los piezómetros dentro del núcleo impermeable a diferentes elevaciones en sitios próximos a la obra de toma, existían tramos en donde se encontraba arcilla en estado líquido, mezclada con algunos fragmentos de roca, similar a la utilizada en la construcción de los respaldos y filtros de la cortina.

Por otro lado, las variaciones piezométricas mostraron que en los piezómetros instalados dentro del núcleo impermeable y que estaban conectados con las zonas antes indicadas, las elevaciones piezométricas eran ligeramente inferiores a las del embalse; sin embargo, todos los piezómetros instalados en la roca de cimentación, sus niveles piezométricos eran 6.0 m a 8.0 m por debajo del nivel del embalse.

Por lo anterior, se concluyó que el fenómeno que dio lugar a los hundimientos durante el primer llenado, correspondía a un efecto de erosión interna, originado por fracturas del núcleo impermeable, debidos a los hundimientos diferenciales en la zona del tajo de desvío, que dieron lugar al inicio de un fracturamiento hidráulico, el cual no pudo ser obturado por la falta de calidad de los materiales con que fue construida la cortina de la presa.

4 ANALISIS DEL FILTRO

4.1 *Características granulométricas del filtro*

Atendiendo a los resultados de la investigación de las causas de los hundimientos antes descritos brevemente, se estableció la necesidad de efectuar el análisis del comportamiento del filtro de la cortina de la presa "El Batán", con el propósito de establecer si fue propuesto adecuadamente, si se construyó de manera apropiada o si requiere algún tratamiento específico.

Para el análisis de este filtro se dispuso de las curvas granulométricas del filtro colocado en la cortina de la presa y del material del suelo base con el cual se elaboró el corazón impermeable.

4.2 *Análisis del filtro con el criterio de Terzaghi*

Este criterio puede catalogarse como conservado, puesto que invita al uso de partículas cuya granulometría sea muy cercana a la del suelo base, es

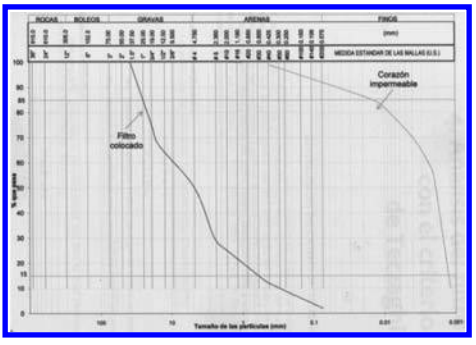

Figura 8. Curva granulométrica del suelo base y del filtro de la presa "El Batán".

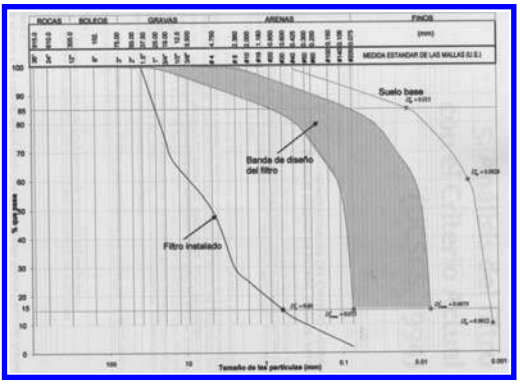

Figura 9. Banda de diseño del filtro aplicando el criterio de Terzaghi.

decir que la granulometría del filtro esté dada en función de la granulometría dl suelo base, siendo éste el único criterio a considerar. Para la determinación de los límites de dicho criterio, se deben obtener 2 valores de acuerdo a las funciones que debe cumplir un filtro. Se presenta en la siguiente figura el resultado de la aplicación de este criterio al filtro de la presa en análisis.

4.3 *Análisis del filtro aplicando el criterio del UCSCS*

Este criterio está basado en los resultados de una investigación realizada en laboratorio, acerca del funcionamiento de los filtros en cortinas de presas, dado a conocer por The Soil Conservation Service at the Soil Mechanics Laboratory en Lincoln, Nebraska, Estados Unidos, entre 1980 a 1985.

El estudio de filtros en el laboratorio demostró que los filtros diseñados con este criterio sellaban sus fisuras. El sellado comienza cuando el flujo de agua a través de las grietas transporta partículas erosionadas hacia las aberturas. Las partículas del filtro erosionado sellan las grietas. Los filtros se

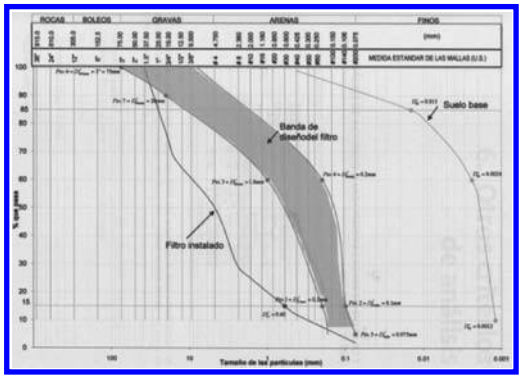

Figura 10. Banda de diseño del filtro de la cortina mediante el criterio UCSCS.

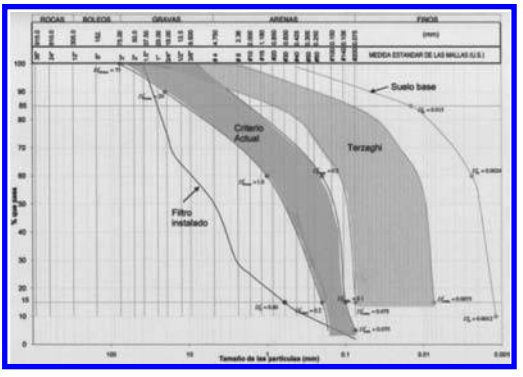

Figura 11. Bandas de diseño del filtro de la cortina de la presa "El Batán".

diseñan para interceptar las grietas, la permeabilidad requerida en la zona dl filtro se debe al constante flujo a través de los poros del suelo base. La capacidad hidráulica de las grietas no necesita ser considerada en el diseño de los filtros ya que éstos mostraron su capacidad de sellarse. Este criterio está establecido también para prevenir movimiento de los materiales del filtro.

Se aplicaron los 13 pasos propuestas en esta metodología del UCSCS al filtro de la cortina de la presa "El Batán" y se obtuvo el resultado para la banda de diseño del filtro presentado en la siguiente figura.

4.4 *Análisis del filtro aplicando otros criterios de análisis*

Finalmente se analizó el filtro de acuerdo a las observaciones de Sherard y Dunningan (1989), aplicándose también el criterio de Foster y Fell (2001) para determinar la frontera no erosionable. Para ambos casos se determinó que el tipo de suelo analizado no experimentó una erosión excesiva, sino que inmediatamente se presenta una erosión continua.

5 CONCLUSIONES

A partir de los resultados mostrados en la banda de diseño del filtro aplicando el criterio de Terzagui (figura 8), se observa claramente que el filtro utilizado en la construcción de la cortina de esta presa se localiza fuera de la zona de diseño.

Asimismo una vez que se ha trazado la franja de diseño utilizando el criterio del UCSCS y se compara con la curva granulométrica del filtro instalado en la cortina de la presa "El Batán", se observa notoriamente como ésta se encuentra fuera de dicha franja. También se aprecia el hecho de que la tendencia que sigue la curva granulométrica del filtro en operación es opuesta a la de la banda de diseño.

Después de realizar el análisis, por medio de diferentes criterios, se concluye que la cortina de la presa "El Batán", ubicada en el municipio de Corregidora, Querétaro en el país de México, falló debido a una mala elección del filtro, según se aprecia en conjunto en la siguiente figura.

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Selecting strength input parameters for structural analysis of aging concrete dams

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ABSTRACT: The Bureau of Reclamation Dam Safety Program performs periodic examinations and risk analysis of all of their dams. Structural analysis results are typically used during a risk analysis to estimate the failure probabilities of concrete dams under static, hydrologic, and seismic loadings. The probability of failure is estimated, often considering average, high, and low strength properties of mass concrete. Not all dams have a complete history of construction and the strength properties must be assumed. A database of mass concrete core tests from the past 50 years was analyzed to determine bond strength input parameters. The average direct tension and shear properties of lift lines were compared for different state-of-the-art construction methods in the 20th Century. The properties were estimated for three primary construction eras and separately for dams suffering from alkali-aggregate reaction. In addition to bond strength, the percent of bonded lift lines greatly influenced the input parameters for these construction eras.

1 INTRODUCTION

The Bureau of Reclamation (Reclamation) is responsible for about 370 storage dams and dikes that form a significant part of the water resources infrastructure of the western United States. As owner of these facilities and in accordance with the mission of the Dam Safety Program, Reclamation is committed "to ensure that Reclamation facilities do not present unreasonable risks to the public, public safety, and/or the environment." Reclamation has established a risk-based framework to assess the safety of their structures, to aid in making decisions to protect the public from the consequences of dam failure, to assist in prioritizing the allocation of resources, and to support justification for risk reduction actions where needed. Structural analysis results are typically considered during the risk analysis to better assess the performance of concrete dams under static, hydrologic, and seismic loads and the likelihood of a structural failure resulting in the uncontrolled release of water and loss of life. Concrete materials properties are needed for structural analysis programs. In some cases, the materials properties for a given concrete dam are unknown. Concrete materials properties are estimated from the Aging Concrete Information System (ACIS) database of historical mass concrete core test results (Dolen 2005). Since the properties of mass concrete dams evolved throughout the 20th Century, the estimates must reflect the state-of-the-practice at the time of construction. Therefore, the average properties are estimated from test results of dams with comparable construction methodology and quality. In addition, the range in materials properties can be used to better estimate the range of risk of structural failure. Of particular interest is the range in tensile and shear properties of lift lines. Estimating the materials properties of mass concrete is only a part of the overall risk analysis and risk assessment program.

1.1 *Risk-based framework for dam safety decision making*

Risk is defined as the probability of adverse consequences, and is estimated as the product of the loading probability, the probability of failure given the load and the consequences of that event. The annualized loss of life is estimated from the product of the annual loading probability and the probability of failure given the load. Reclamation identifies potential failure modes for each dam in our inventory and estimates annual probability of failure and risk for plausible failure modes that could result in an uncontrolled release of the reservoir. Risk analysis is the quantative calculation of risk. Risk assessment is performed at a given dam once a risk analysis is completed and is the process of deciding whether risk reduction actions are needed. Annual probability of failure estimates and risk estimates are compared to Reclamation's Public Protection Guidelines as part of risk assessments (Reclamation 2003). For a given dam, Reclamation considers potential failure modes in three different

loading categories—static, hydrologic, and seismic, and risk are estimated over a full range of loading conditions for each potential failure mode. Reclamation may take measures to reduce risk and/or to better quantify the uncertainties associated with the risk before taking action. Reclamation considers risks greater than 0.01 lives per year to require expedited action to reduce risk. For risk estimates between 0.01 and 0.001 lives per year and for annual probabilities greater than 0.0001, there is justification for taking action to reduce risk, normally within the scheduled dam safety program budget or evaluation process. For risk estimates to be less than 0.001 lives per year and for annual failure probability estimates less than 0.001 there is diminishing justification to implement risk reduction measures, or actions considered reasonable and prudent are implemented.

1.2 *Risk analysis and structural analysis*

For many potential failure modes related to concrete dams, a structural analysis provides valuable information on stresses and stability of the structure. During risk analysis, the probability of failure for each failure mode is estimated, based on the risk team's evaluation of a structural analysis using assigned materials properties of mass concrete.

1.3 *Strength input parameters for risk and structural analysis*

Reclamation became concerned that estimates of materials properties and the range of these estimates were significant factors when estimating the risk of dam failure. In some cases, the range of estimates for mass concrete strength was enough to produce a wide range of potential risk. Reclamation began a research program to better identify the materials properties of mass concrete and in particular, the bond strength of concrete at horizontal construction joints, or "lift lines." Lift lines are planned interruptions in the placement, the concrete has chemically "set" and gained nominal strength before the next placement. Loosely defined, the lift lines are nearly horizontal concrete surfaces resulting from planned work stoppages, usually corresponding to the end of a work day or shift, or topping off formed placements. Lift lines usually must be cleaned to remove top surface "laitance" and debris just before placing the subsequent lift to assure impermeability, adequate bonding, and monolithic behavior. Classic lift lines are the regularly spaced construction joints resulting from topping off a formed placement. The bond strength of lift lines normally represents the weakest concrete from a strength perspective.

1.4 *ACIS database*

The ACIS database is a relational database of historical mass concrete tests. The database was originally created to investigate the properties of aging concrete compared to other like concretes without aging (Dolen 2005). Concrete data can be categorized by date, structure, mixture properties, specimen size, and various aging mechanisms for later sorting. For this investigation, the database was modified to add individual direct tension and direct shear tests, including failure mechanisms. Figure 1 shows an example histogram of direct tension strength tests showing the distribution of lift line tests that failed at the lift line bond, ave of 1050 kpa, and those that failed away from the lift line, ave of 1330 kpa. Figure 2 shows the distribution of direct tension tests of parent (no lift line) concrete that failed near the center of the specimen, ave of 1470 kpa, and those that failed near end plates, ave of 1530 kpa. Neither the center, nor near-platen failure histograms follow a typical normal distribution. This is attributed to combined data from different dams with both low strength and high strength concrete in the same sample populations.

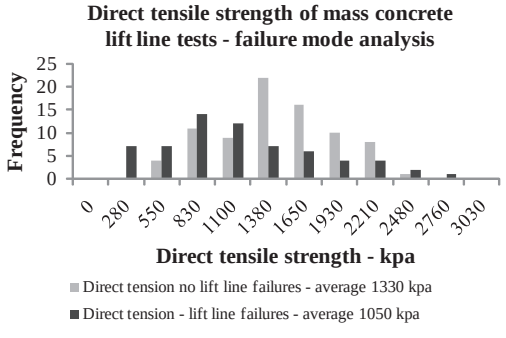

Figure 1. Histogram of direct tensile strength of mass concrete lift lines showing failure modes.

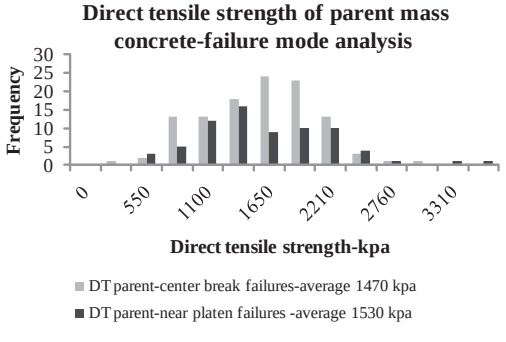

Figure 2. Histogram of direct tensile strength of parent mass concrete showing failure modes.

Based on the failure mode analysis, lift line versus no lift line test failures were considered separate data populations, whereas the parent center and end plate failure mechanisms were combined into a single population. From these overall sample populations, generational subgroups were identified based on the year of construction and lift line surface preparation methods. When sorted by specimen diameter, there was little difference between the average direct tensile strength of 150 mm diameter tests compared to larger diameter test specimens, 225 to 460 mm diameter (for each characteristic population).

The individual shear, break-bond tests were added to the database to produce subgroups based on the year of construction and lift line surface preparation methods. The data includes the test normal stress and shear failure strength for both lift line and parent concrete test specimens.

2 TIMELINE FOR CONCRETE DAM CONSTRUCTION AND LIFT LINE SURFACE PREPARATION METHODS

A historical timeline was developed to identify bond strength input parameters for dams constructed in different eras by different lift surface cleaning and placement methods. Reclamation analyzed construction records, drilled core logs, and test results to determine the average bond strength properties for different construction methods.

2.1 *Lift surface cleaning and preparation methods*

Reclamation concrete dam evolution followed the rapidly developing state-of-the-practice in mass concrete construction throughout the 20th Century. Initially, dams were constructed using relatively rudimentary techniques, equipment, and materials. As dams became larger and more complex, the construction techniques also evolved. The construction of Hoover Dam from 1933 to 1935 led to a vast advancement in mass concrete technology. Figure 3 shows a historical timeline of the evolution of lift line surface cleaning methods.

The properties of lift lines were affected by the available equipment, the speed of construction, ambient temperature and humidity, and the strength of the underlying concrete. There is a narrow lift line cleaning "window" from about 8 to 96 hours where the concrete strength is low enough to be effectively cleaned by the available equipment and methods before placing subsequent lifts of concrete. As the strength of concrete increased, the window of opportunity for effective lift surface cleaning decreased. New equipment and methods were therefore needed to compensate

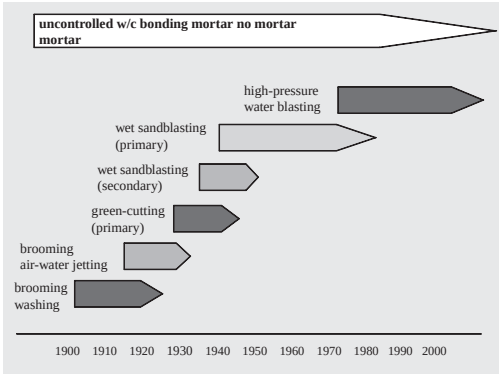

Figure 3. Timeline of Reclamation concrete dam lift line surface cleaning methods.

for the ever-increasing strength and performance requirements. About five types of lift line surface cleaning methods were identified;

- wire brooming and washing,
- brooming and air-water jetting,
- early age "green-cutting,"
- delayed, wet sand blasting, and
- high-pressure water blasting.

There is some overlap between when one method was abandoned in favor of a more cost efficient and effective method.

2.2 *Aging concrete dams*

The properties of aging concrete dams form a subset of the overall properties of mass concrete. Aging dams include those affected by freezing and thawing deterioration and/or Alkali Aggregate Reaction (AAR). The strength input parameters of AAR affected dams were separated from the properties of unaffected dams.

3 CONCRETE CORING AND TESTING

Reclamation has drilled hundreds of meters of concrete cores from dams up to 100 years age. These cores are tested for a variety of strength parameters. Core drilling and test methods have evolved over the years. Test specimens themselves can be subject to sample bias, depending on the specific purpose of the core drilling program. In some cases, the drilled cores are biased to investigate specific areas of poor quality in a dam, perhaps in areas of low strength or deterioration. The test data was carefully examined before summarizing strength input parameters. The test averages represent primarily vertically drilled cores tested in the saturated moisture condition.

3.1 *Estimating the percent bonded lift lines*

Without incorporating the percent bonded lift lines, bond strength test results represent a somewhat biased sample record. An un-bonded lift line cannot be tested for strength, only friction parameters. Thus, the results must be evaluated to determine if they are truly representative of the overall dam quality. A drill core log represents the best available information on the integrity of lift lines. The percent of bonded lift lines was estimated based on the available records from the number of lift lines obtained intact compared to the total number of lift lines intercepted. Figure 4 shows the change in percent of bonded lift lines over time. The moving average was used to smooth trends. One factor affecting lift line bond strength was placing overly wet mixtures with excessive bleeding and laitance. Additional water was added to transport concrete in long chutes and to consolidate the mass concrete in higher lifts. Introduction of the highline bucket method of placing mass concrete about 1930 and the internal mass concrete vibrator in 1934 allowed placing lower slump concrete, with a subsequent improvement of lift line properties. In addition, Reclamation required placement of about a 10 to 20 mm thick layer of mortar broomed into the cleaned lift line surface immediately before placing mass concrete up to about 1965. The average properties with and without mortar were compared where appropriate.

Broken lift lines must be examined carefully to determine if they are un-bonded or caused by mechanical breaks. The percent of bonded lift lines was considered a significant factor in overall mass concrete dam performance, and can be correlated to better lift line cleaning methods. The average and range of percent bonded lift lines are summarized in Table 1.

There is more uncertainty if only a single core is drilled from a dam than from multiple spaced drill holes which provide a more global estimate of the percent of bonded lift lines. If multiple core holes indicate there is a continuous, un-bonded lift line, additional analysis should evaluate this condition.

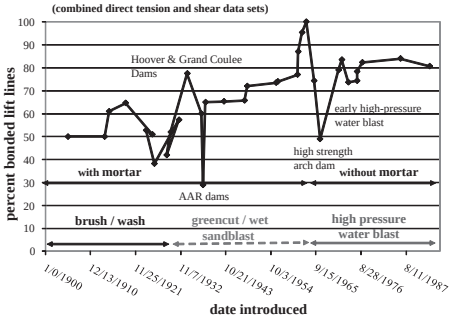

Figure 4. Moving average of estimated percent bonded lift lines for mass concrete dams constructed from 1905 to 1993.

Table 1. Estimate of percent bonded lift lines observed from drilled cores of Bureau of Reclamation concrete dams.

	Average bond	Range low	Range high
Year constructed	Percent	Percent	Percent
1905–1933	50	9	83
1933–1964	85	66	100
1965–1993	74	23	96
1965–1993*	83	66	96
AAR** 1925–1938	40	12	89

*Average for 1965 to 1993 not including early development of high-pressure water blasting, lift surface cleaning technology; **Alkali aggregate reaction affected dams constructed between 1925 and 1938.

From these estimates, the designer may choose a more conservative estimate of the direct tensile strength or cohesion of bonded lift lines by reducing the average values by the estimated percent of bonded lift lines. Or, appropriate bond strength values could be applied to bonded lift lines and apparent cohesion applied to un-bonded lift "contact surfaces," based on the estimated percent bonded lift lines. The more conservative approach is to perform analysis assuming the lift lines are un-bonded with only apparent cohesion and sliding friction resistance. The estimated percent bonded lift lines are added with the average values for shear and tension tests in the ACIS database.

Early Reclamation dams averaged about 50 percent bonded lift lines until about 1930. After Hoover Dam, the average percent bonded lift lines increased to about 80 percent. The increase in percent bonded lift lines after Hoover Dam may be the result of several factors including:

- introduction of controlled, green-cutting and/or wet-sand blasting,
- increased overall concrete strength,
- improved quality control methods and inspection, and
- introduction of internal vibrators for consolidating concrete.

A few dams had noticeably lower intact lift line recovery including several AAR affected dams, dams with high early strength concrete, and the first dam to use high-pressure water blasting as the primary lift line surface cleaning method.

3.2 *Sample selection and bond strength properties*

This study analyzed the direct tensile and biaxial shear tests of "parent concrete" (no lift lines) and of lift lines. Laboratory tests of parent concrete representing the in-situ properties of the mass concrete. Lift lines are normally centered in the test specimen. The shear and tensile properties of good quality concrete and construction methods approach the properties of the parent mass concrete. Poor quality concrete and construction practices significantly lower lift line properties compared to parent concrete. The ratio of strength between lift lines and parent concrete gives an indication of the quality of construction.

3.3 *Direct tension test*

The direct tension test is performed according to USBR No. 4914, "Procedure for Direct Tensile Strength, Static Modulus of Elasticity, and Poisson's Ratio of Cylindrical Concrete Specimens in Tension" (Reclamation 1992). The current test method, shown in Figure 5, uses double–bolted, steel platens bonded to each end of the specimen with epoxy resin to distribute the tensile stress evenly at the platen—concrete interface. The results of parent concrete and lift line tests should be used cautiously since they normally represent the better quality concrete of testable size, and only the bonded lift lines. During a test program, the direct tension lift line specimens are examined to determine if the concrete in fact failed at the lift line surface, or at another plane of weakness. In some cases, the strength of lift lines may be quite adequate, but a low percentage of bonded lift lines will significantly bias the analysis.

Figure 5. Direct tensile test performed on 450 mm diameter drilled concrete core using double-bolted, steel end plates.

3.4 *Bi-axial, direct shear test*

McLean and Pierce described the current practices of Reclamation bi-axial shear strength testing according to USBR No. 4915, "Procedure for Direct Shear of Cylindrical Concrete Specimens" (McLean and Pierce 1988), (Reclamation 1992). Four data sets may be obtained from these tests:

- break-bond test cohesion and coefficient of internal friction of "parent" mass concrete,
- break-bond test cohesion and coefficient of internal friction of bonded lift lines,
- apparent cohesion and sliding friction resistance of the broken surfaces of the parent concrete or, of lift lines after initial break bond failure, and
- apparent cohesion and sliding friction resistance of "un-bonded" lift lines.

Several specimens are required for each sample set to determine the cohesion and internal friction angle of parent concrete and bonded lift lines. Three or more "break-bond" specimens are tested with different (constant) normal loads until there is a failure, or large horizontal displacements of the specimen are recorded. A graph of each normal versus shear strength test specimen is plotted and a linear, "best fit" curve is used to determine the cohesion, C (intercept at zero normal stress) and the internal friction angle, ϕ as shown in Figure 6.

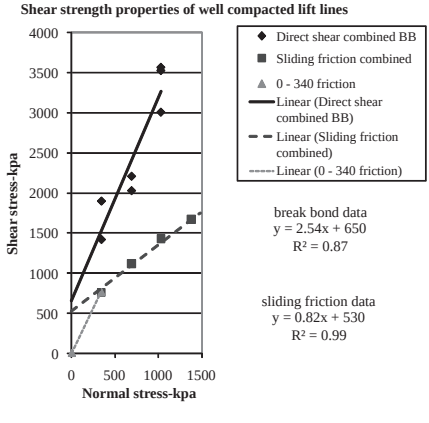

Figure 6. Example combined plots of shear break-bond and sliding friction data sets.

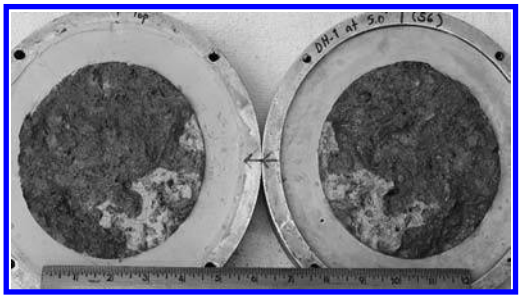

Figure 7. Lift line surface after direct shear, break bond test.

The coefficient of internal friction, tan ϕ, is obtained from the slope of the linear regression equation. For each broken or un-bonded lift line test specimen, three to six sliding tests are performed on each specimen with the normal stress increased after large displacements are recorded. Apparent cohesion, C_a is the projected intercept of the linear best fit equation at zero normal stress; obtained from sliding tests of broken surfaces, or of un-bonded lift lines. Phi_a is the sliding friction angle obtained from the slope of the linear regression equation of sliding tests. A second sliding equation is also projected from the origin of the graph 0,0 kpa up to the first recorded sliding test. This would be used assuming no apparent cohesion, but has a steeper friction angle. The broken shear specimen surfaces, Figure 7, are examined to investigate surface asperities and/or possible encapsulating cement interference.

3.5 *Individual versus weighted average properties*

The individual direct tension and shear test results were added to the database for each dam. The bond strength averages are based on all tests representing each subgroup. Weighted averages were calculated for compressive strength and elastic properties, and splitting tensile strength based on the average strength weighted by the number of tests. That way, the average strength for many tests from one dam outweighs those where only a few specimens were tested.

4 BOND STRENGTH INPUT PARAMETERS

The following tables summarize the input parameters for direct tensile and shear strength of mass concrete for use in risk analysis. The average properties and the range are suggested for estimating risk for most projects. The range is shown as plus or minus one standard deviation from the average strength. The lowest values are used for "worst case" conditions. However, the estimated percent of bonded lift lines, Table 1, is critical to using strength input parameters estimates correctly for risk analysis. Dams constructed before about 1933 have a higher probability for many un-bonded lift lines and the potential for a completely un-bonded lift line across the entire dam section.

Dams suffering from alkali-aggregate reaction have differing strength input parameters. In addition, these dams have the potential for a significant percentage of un-bonded lift lines. Even AAR dams with high lift line bond, where the concrete has little reaction, show degrading lift line strength where the reaction is more severe (Joy 2010). In these instances, the dam should be analyzed assuming no tensile strength across lift lines.

4.1 *Direct tensile properties of parent mass concrete and bonded lift lines*

The direct tensile properties of parent mass concrete and bonded lift lines are summarized in Table 2. This table includes the average properties for all direct tension tests, followed by the average properties for three construction eras, 1905 to 1933, 1934 to 1965, and 1965 to 1993. The lift line averages of sub-groups represent those tests that were identified as having failed at the lift line. The average lift line strength is about 80 percent of the parent concrete strength. The average strength of lift line specimens that did not fail at the lift line interface is about 90 percent of the parent mass concrete strength.

Table 2. Direct tensile strength of parent mass concrete and bonded lift lines (no alkali aggregate reaction).

	Average strength	Standard deviation	Minimum	Number of tests
	kpa	kpa	kpa	
All parent mass concrete	1460	530	280	197
1905–1933	1080	420	410	19
1934–1964	1530	540	280	53
1965–1993	1510	530	480	112
All bonded lift lines	1200	550	70	154
1905–1933 [50]*	700	530	70	28
1934–1964 [85]	1130	490	480	16
1965–1993 [74]	1340	570	480	27

*[Estimated percent bonded lift lines for each subgroup].

4.2 *Direct shear properties of parent mass concrete and bonded lift lines*

The average direct shear properties of parent mass concrete and bonded lift lines are summarized in Tables 3 and 4, respectively. Example data sets for tests of parent concrete and bonded lift lines are shown in Figures 8 and 9, respectively. The average cohesion and internal friction angles are derived from a linear regression of all tests for specific subgroups. This may include both stronger and weaker concrete in the same data set, resulting in very low correlation coefficients. The "90 percent exceeding" cohesion values were obtained by projecting the average linear regression line with the average phi angle to the point where 90 percent of test values were above it. The average cohesion of all bonded lift lines is similar to the direct tension results, about 80 percent of the average parent mass concrete strength.

The data representing parent concrete from dams constructed between 1905 and 1933 show an unusually low phi angle. Another estimate for cohesion was found by projecting a dashed line with a 45 degree slope through the 1905 to 1933 data in Figure 8.

The internal friction angle of bonded lift lines for dams constructed between 1905 and 1933 is also low. This could have been caused low strength concrete at lift lines and related to early 20th Century placing techniques using the "chuting" transport method. Overly wet mixtures with slumps of 150 mm or more were used to facilitate transporting and consolidating concrete in the absence of suitable mechanical equipment. These lift surfaces had excessive bleed water, and little aggregate interlock due to settlement.

The shear strength of early 20th Century dams is about 70 to 80 percent of the strength of "post Hoover" dams. There were two notable changes

Table 3. Direct shear, "break-bond" properties of parent mass concrete, 1905 to 1993.

	Average cohesion	90 percent exceeding	Internal friction angle, ϕ	Number of tests
	kpa	kpa	degrees	
All parent mass concrete	3960	1100	48	25
1905–1933 [estimated]*	3550 [3000]	1450 [1000]	12 [45]	8
1934–1964	3410	2410	61	11
1965–1993	4100	3450	75	6
1934–1993	4380	3030	53	17

*1905 to 1933 data extrapolated with phi equal to 45 degrees.

Table 4. Direct shear, "break-bond" and sliding friction properties of bonded mass concrete life lines, 1905 to 1993.

	Average cohesion	90 percent exceeding	Internal friction angle, ϕ	Number of tests
	kpa	kpa	degrees	
All bonded lift lines	3240	1100	49	107
1905–1933 [50]*	2860	1720	32	46
1934–1964 [85]	3480	1720	54	35
1965–1993 [74]	3960	2620	53	26
1934–1993 [80]	3730	2340	53	61
Sliding friction 1905–1993	(C_a) 500		(ϕ_a) 46	130

*[Estimated percent bonded lift lines].

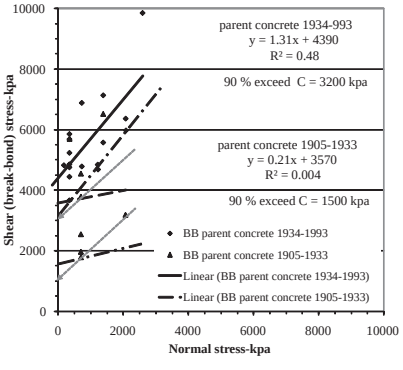

Figure 8. Shear break-bond properties of parent mass concrete from 1905 to 1933 and from 1934 to 1993.

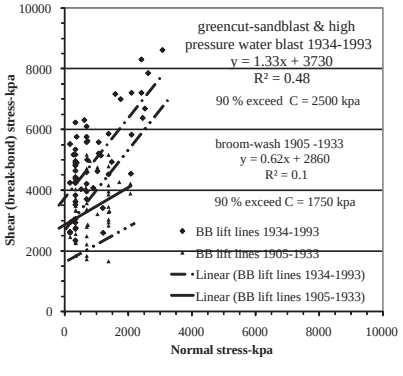

Figure 9. Shear, break-bond properties of bonded mass concrete lift lines; effects of different lift surface cleaning methods.

in lift placing methods after about 1930. Mass concrete mixtures were placed using the overhead highline and bucket method after 1930 and the first internal vibrators were introduced for mass concrete consolidation about 1934. These two changes led to noticeably superior concrete. Mass concrete could now be placed at a slump of about 50 mm; the reduced water content decreased the paste volume, resulting in less surface laitance from bleeding and less aggregate settlement. Coupled with the introduction of more efficient wet sand-blasting, lift lines now had a more undulating profile with better aggregate interlock.

The 1934 to 1964 and 1965 to 1993 data represent lift lines with and without a 20 mm layer of bonding mortar spread over lift surfaces. For convenience, Figures 8 and 9 compare the 1905 to 1933 data to the combined 1933 to 1993 mass concrete test data for parent concrete and for bonded lift lines, respectively.

4.3 *Lift line properties of alkali aggregate reaction affected dams, 1925 to 1938*

Lift line properties for dams suffering from AAR were noticeably lower than comparable dams of that generation. Table 5 summarizes the average direct tensile properties of mass concrete suffering from AAR. The tests are compared to other dams constructed during the same time frame not suffering from AAR. The average direct tensile strength of AAR-affected parent concrete is about 40 percent, of the non-affected concrete. Lift line strength of AAR-affected dams is about 30 percent of comparable non-affected dams. It should be noted the few tests performed on bonded lift lines is indicative of the low percent of testable, bonded lift lines recovered from AAR affected dams. This may be the result of both AAR and/or the placing conditions during construction (in arid climate). A few dams suffering from AAR with comparatively good lift line bond also seemed to be de-bonding in specific areas with the more severe reaction and expansion. Table 6 shows the shear and sliding friction properties of lift lines for AAR affected dams compared to un-affected dams built between 1925 and 1938. The cohesion is about 70 percent of comparable concrete and the internal friction angle is reduced by about 10 percent.

Table 5. Direct tensile strength of parent mass concrete and bonded lift lines with and without alkali aggregate reaction.

	Average strength	Standard deviation	Minimum	Number of tests
	kpa	kpa	kpa	
AAR parent mass concrete 1925–1938	690	340	70	81
AAR bonded lift lines [40]*	480	410	210	9
No AAR** parent mass concrete 1925–1938	1550	580	620	9
No AAR** bonded lift lines [80]	1675	685	690	17

*[Estimated percent bonded lift lines]; **Results of tests from comparable dams with no AAR.

Table 6. Direct shear, "break-bond" and sliding friction properties of AAR affected lift lines and comparable unaffected mass concrete, 1925 to 1938.

	Average cohesion	90 percent exceeding	Internal friction (phi)	Number of tests
Lift lines	kpa	kpa	degrees	
AAR 1925–1938 [40]*	2520	1380	49	28
No AAR 1925–1938 [80]	3480	1900	56	23
AAR sliding friction 1925–1938	(C_a) 440		(phi_a) 47	88

*[Estimated percent bonded lift lines].

4.4 *Weighted average strength properties*

Table 7 summarizes the weighted average compressive strength and elastic properties of mass concrete dams for different construction eras. The weighted average takes into account the average strength for various test programs weighted by the number of tests for each program. The average compressive strength and elastic properties of early 20th Century dams is about two-thirds of those completed after Hoover Dam, indicating better quality concrete and improved construction methods. The properties of AAR affected dams are compared to un-affected dams constructed from about 1925 to 1938. The average compressive strength and modulus of elasticity of AAR affected concrete is about 30 percent lower than un-affected mass concrete. In many cases, the AAR data is biased with better quality concrete due to the inability to test severely deteriorated concrete.

The weighted average splitting tensile strength of parent mass concrete is about 2870 kpa, and ranges

Table 7. Weighted average compressive strength and elastic properties of mass concrete, 1905 to 1993, and with or without alkali aggregate reaction, 1925 to 1938.

	Average strength	Modulus of elasticity	Poisson's ratio	Number of tests
	mpa	gpa		
All concrete	31	28	0.19	536
1905–1933	24	21	0.20	227
1934–1964	35	36	0.19	185
1965–1972	41	33	0.16	63
1973–1993	37	31	0.16	63
AAR dams 1925–1938	29	22	0.19	399
No AAR dams 1925–1938	40	32	0.23	23

from about 2600 kpa for dams constructed before 1933 to about 3550 kpa for dams constructed after about 1933. The weighted average splitting tensile strength of AAR affected dams was about 2340 kpa, about 25 percent lower than un-affected dams built between 1925 and 1938, about 3100 kpa.

4.5 *Roller-compacted concrete dams*

Roller-Compacted Concrete (RCC) dams form another subgroup of dams constructed after about 1981. There is less data available on cores obtained from RCC dams. Generally, the bond strength of RCC dams, particularly early RCC dams is significantly affected by compaction at the bottom of lifts (Drahushak-Crow 1988). Data is currently being compiled for RCC dams, where available.

5 CONCLUSIONS

Risk evaluation is an accepted practice for dam safety decision making. Structural analysis is used during a risk analysis to estimate the probability of failure for a given event. The range of probability of failure was found to depend on the range of concrete materials properties assumed in the analysis. A database of concrete materials properties was examined to better estimate the average bond strength and the range of strength for dams constructed between 1905 and 1993. The data analysis shows there is sufficient justification to apply separate strength input parameters in risk analysis for dams constructed with different state-of-the-art lift line preparation methods. Both the percent of bonded lift lines and the average and range of strength are considered significant. Of particular interest is the difference in bond strength properties of early 20th Century dam construction methods compared to improved methods developed for Hoover Dam. The introduction of the internal vibrator for consolidating mass concrete is under appreciated with regards to improved lift line performance. The average and range of properties of AAR affected dams are also noticeably lower than comparable dams with similar construction methods.

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Rehabilitation of Tuilières Dam: Behaviour under seismic load

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ABSTRACT: The Tuilières dam is a masonry gate-structure dam built in 1908 on the Dordogne River, 20 km ahead from Bergerac. 9 masonry piers and 8 Stoney gates compose the gate-structure part of the dam. During the night of the 29th of January 2006, the gate n°4 breaks and the reservoir emptied. This paper presents the rehabilitation of the dam following this incident and more particularly the analysis of the behavior under seismic load, which leaded to the realization of a new bracing.

1 PRESENTATION OF THE DAM

The Tuilières dam is composed by, from left bank to right bank:

- a fish passage,
- a masonry gate-structure dam, (subject of this study),
- the powerhouse.

It's been built in 1908 on the Dordogne River, 20 km ahead from Bergerac.

The gate-structure part of the dam, presented on figure 1, is made of 9 masonries piers. The first sluice on the right side is 7 m wide; the others are 10 m wide. Theses sluices are closed by Stoney gates, vertically moved by a system of winches, located on a platform, on the top of the piers at elevation 49.00, and counterweights.

A reinforced cement footbridge at elevation 36.00 is used to cross the dam and assumes in addition a rule of bracing. The main characteristics of the piers are:

- thickness: 3 m (except pier n°2: 4 m),
- height: 31.30 m,
- length upstream-downstream: 16.65 m.

Figure 1. Gate-structure dam after the incident.

The piers are built on a thin apron made of concrete with a cut off. There is no drainage system.

2 THE INCIDENT AND ITS CONSEQUENCES

During the night of the 29th of January 2006, the gate n°4 broke and the reservoir emptied. The dam was then placed in "revision spécial" by the authorities: that decision includes the need to present to them a rehabilitation project and to prove the stability of the rehabilitated dam.

Despite the low seismicity of the site but because of the great height of the piers, compare to their thickness, EDF impose itself to check the stability of the dam under seismic load in addition of the respect of the usual stability criteria.

In order to reduce the delay before filling again, the decision to keep the dam as close as possible of the original design was taken.

3 STABILITY ANALYSIS OF THE DAM BEFORE REHABILITATION

First of all, a full two-dimensional analysis of a pier has been produced in order to check the stability under normal load, for the 1000-year flood and under seismic load with a pseudo-static analysis. Because of the geometry of the piers, the gate-structure dam is also particularly sensible to side to side solicitation that could occur during a seism. That is not taken into account in the two-dimensional analysis. A finite element analysis of the whole gate-structure dam has then been performed in order to understand the behaviour during such event.

3.1 *Finite elements model*

The finite element model represents the whole gate-structure dam with the 9 piers and their

bracing, the apron and the foundation with about 14,000 solid twenty-noded brick elements and 70,000 nodes. The gates have not been modeled: the water pressure will be directly reported on the piers. The abutments on the left side and on the powerhouse are also taken into consideration. On the top of the pier, elements with adequate density represent the weight of the top-platform, the winches and the counterweights.

3.2 *Dynamic analysis method*

The seismic analysis is realized with Code_Aster using a modal recombination method:

- computation of the elastic eigenmodes,
- selection of the eigenmodes (only the most representatives are take in account),
- computation of the dynamic response in the eigenmodes basis,
- post-processing of the displacements, acceleration, stress on nodes,
- superposition of the stress fields computed from this dynamic analysis and a static analysis (gravity, water load applied on the piers, uplift).

The choice has been made to represent the hydrodynamic load with the Westergaard method: as we are looking for the side-to-side behaviour of the piers, the influence of the hydrodynamic effect is considered weak.

3.3 *Earthquake*

A seismic risk study has been realized in order to determine the earthquake that can affect the dam:

- a deep earthquake, intensity VI-VII MSK and a peak ground acceleration of 0.04 g,
- a shallow earthquake, intensity V-VI MSK and peak ground acceleration of 0.01 g.

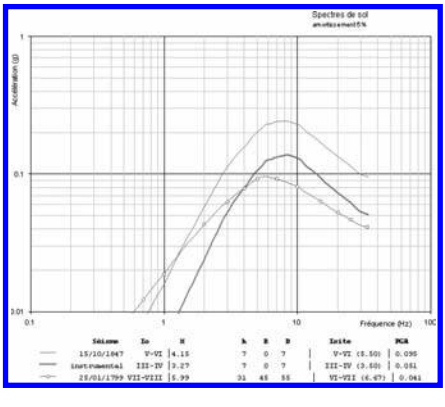

Figure 2. Ground spectrums.

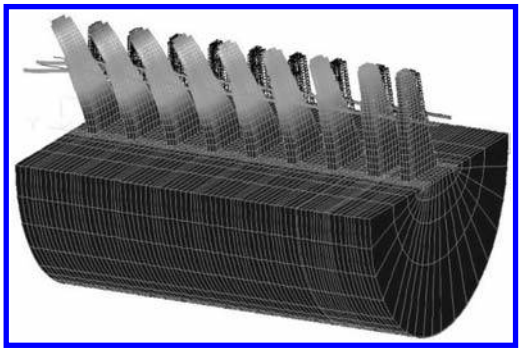

Figure 3. Deformed shape for the first mode.

The last one is confirmed for the analysis as the stronger in the frequencies concerning the dam.

From this spectrum, 3 accelerations signals are generated and combined in order to produce the most unfavorable ground motion for the structure.

3.4 *Eigenmodes*

The modal analysis computes 100 eigenmodes between 0 and 30 Hz. Only 24 are really involving a significant percentage of weight. The first one is obviously a side-to side and synchronized flexure of all the piers, at 2.77 Hz.

The first flexure mode in the upstream-downstream direction occurs for 8.32 Hz, at the peak of the spectrum.

Both of them affect about 40% of the weight of the structure in their direction.

3.5 *Results and interpretation of the analysis*

The dynamic analysis shows that a synchronized and side-to-side oscillation movement of flexure of the piers could probably occur during earthquake. Displacements about 1.2 cm are computed on top of the piers. On the contact pier-apron, tensile stresses of 0.7 MPa are observed, that could lead to a full degradation of the contact.

In such event, the bracing is very important to prevent an increase of the displacement between two pears that can lead to block the gate. But high values of tensile stress (from 2 to 4 MPa) are also computed at the contact between bracing and piers. The masonry won't probably accept such values and the link between bracing and piers will probably break.

4 REHABILITATION PROJECT

As presented before, the rehabilitated dam as to be as close as possible of the original design.

The purpose is then to improve as best as possible the behaviour under seismic load with the lowest modifications. The rehabilitation project brings also some transformation of the gate-structure dam according to others needs presented on the following chapter.

4.1 *Other aspects*

Here will only be described the modifications of the gate-structure dam imposed by others reasons than the improvement of the behaviour under seismic load. Nevertheless, theses modifications have to be taken into account in the analysis:

- the noses of the piers are extended upstream in order to improve the way to install stoplogs,
- the replacement of the whole system of gates, winches, counterweights and the upper-platform modify the repartition of mass on top of the piers.

This leads to a modification of the finite-elements model and have an influence on the behaviour presented on the followings chapters.

4.2 *Path to a new bracing*

In order to reduce the side-to-side flexure of the piers under seismic load, a continuous reinforced bracing with strong abutments on the powerhouse and on the left side has been imagined. Unfortunately, such solution leads to an increase of the first eigenmode in frequency. As the peak of the spectrum is around 8 Hz, under this value, an increase of the eigenfrequency leads to an increase of the acceleration. In consequences, such bracing doesn't reduce the displacements on top of piers; moreover it increases the force on the abutment.

Others analysis show that the release of the abutment on the side of the gate-structure leads to a decrease of the first eigenfrequency and a reduction of displacements on top of piers.

Finally, a new bracing is designed 3 m above the level on the old one. This raising contributes also to lower the first eigenfrequency. The first eigenmode of the gate-structure is now about 1.93 Hz with the followings effects on the dynamic results:

- side to side max displacements on top of piers: 0.6 cm (instead of 1.2 cm before),
- effective tensile stress at the contact pier-apron: 0.17 MPa (instead of 0.7 MPa),
- tensile stress at the contact pier-bracing: 0.6 MPa (instead of 2.7 MPa before).

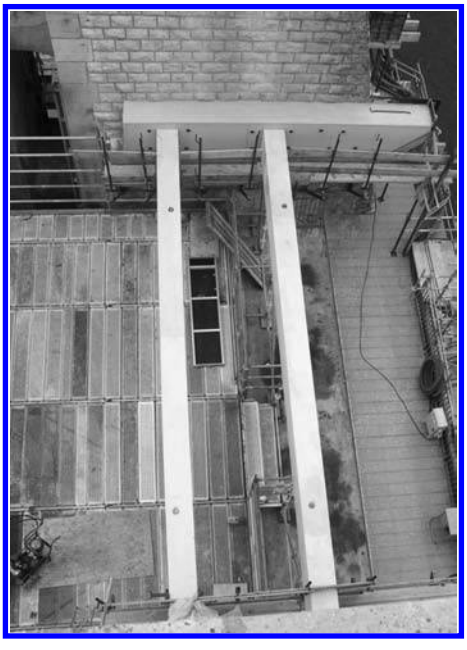

Figure 4. The new bracing.

5 CONCLUSION

Despite the strong restrictions on the design imposed by the decision to rebuilt the dam as before, the improvement of the side to side behaviour of the gate-structure dam under seismic load has been reached. The chosen solution is well integrated and contributes to a good geometric reinforcement of the piers in case of strong ground motion. However, the modification doesn't change anything for the behaviour in the upstream-downstream direction: the analysis shows a risk of cracking at the contact pier-apron.

One of the weak points of this study is the choice of the material properties. They have only been chosen through results of mechanical tests. Common values of damping factors were used. But as the purpose was to improve the behaviour, only a global comprehension of this behaviour was searched and the direction to improve it (here: lower the eigenfrequencies).

This study confirms the sensibility of this type of dam composed of slender piers with heavy mass at top, under seismic load. In this case, at top of the piers, accelerations of 0.4 to 0.5 g have been computed: about 5 times the peak ground acceleration.

Remedial and construction techniques

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Rehabilitation for high Concrete Faced Rockfill Dam (CFRD): Availability and vulnerability

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ABSTRACT: Rain fall area, tectonic location, inaccessibility to qualified available earth core and/or impervious materials and economical aspects are all that leads to Concrete Face Rockfill Dam (CFRD) type selection. Design and construction of high and extra high CFRDs have been carried on empirically so far, and recent failures that happened around the world, promoted any facts in this way. Monitoring behavior and realistic FEM analysis must still be accurately implemented to enhance high and extra high CFRDs performance predictions. Safety behavior during operation phase and current speed up construction methods technology are the so called advantages of CFRD, compared to another alternative.

This paper describes on a few rehabilitation projects including 12 CFRDs those causes mostly on structural cracks at the End Of Construction (EOC), concrete face failures due to compress and flexion stress in First Filling Impounding (FFI) and also in intense earthquake period. Concrete slabs and parapet wall parts movements, topography plinth foundation geometry and also high leakage are another problems those remedial treatments are described too. So, rehabilitation in very high CFRD is as value as another advantage for same dam type, produce salient points which is the main purpose to present this paper.

Keywords: extra high concrete faced rockfill dam, cracks, spalling, compression stress, rehabilitation

1 INTRODUCTION

The present paper looks to overview rehabilitation of 12 CFRDs chronologically in end of construction, at the first filling impounding and also in performance period after a few years of operation. Hence, the main purpose of illustration of cracks and failures repairing works in CFRDs is not only to happen probably incident but also for accessible method statement for rehabilitation.

Despite of operational impacts and losing financially and environmentally matters, CFRD repairing works shown be simplified even in any close valley types, comparatively as another dam type. It should be added that exposed face slab waterproof element increases the impact risk otherwise might be considered as an advantage subject in reconstruction and further maintenance works.

2 CRACKS AND RUPTURES CASES

Some cases of harmful effects in high CFRDs are presented, as follows.

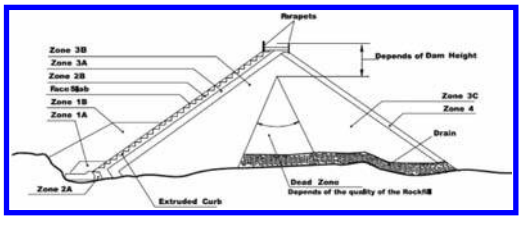

Figure 1. Typical Zoning of CFRD.

2.1 *Aguamilpa*

The Dam starts operation in 1993, in Mexico, 186 m high, 137,000 m^2 face slab area, A/H^2 = 3. (A is the area of concrete face and H is the dam height), where horizontal and diagonal cracks in the slabs were recorded after the filling of the reservoir. A quite range ratio of compressibility moduli of the two materials: in Zone 3B—a compacted dredged alluvial gravel (200–300 MPa), and downstream zone 3C, there was a rockfill material from blasted required excavations (50–60 MPa), was the main reason of the cracks family along the slab. The leakage reached around 260 l/s in Oct. 1994 and

cracks were sealed and leakage was under control during operations phase.

2.2 *Xingo*

The dam completed in 1994, in Brazil, by the height of 140 m, 135,000 m^2 face slab surface, with A/H^2 = 6.9 and max. settlement reported of 2.90 m. Cracks in the surface of Zone 2B back to filling placement and afterwards joints opening was observed up to 300 mm and also cracks in face slab along 8 m with 15 mm in width. Cracks and high stress concentrations in the slab appear to have been produced by the abrupt topography and rock geometry of foundation in the left abutment and also attributed to local sharp change in plinth alignment and slope creating. The treatment by dumping silty sand from dam crest just in suction points of concrete slab has caused an effective reduction of flow.

2.3 *Itá*

Itá CFRD in Brazil, with the 125 m high, face slab 110,000 m^2 area, has the valley shape factor around 7, was the first CFRD where the extruded curb method (1999) for upstream slope protection was used. In first filling impounding leakage reached up to 1700 l/s. Underwater survey with a R.O.V. (Remotely Operated Vehicle) found cracks in concrete face, about 10–15 m above and parallel the plinth. Dumping of clay and sand were successful in reducing the leakage from 1700 l/s to 380 l/s. In addition, regrouting works deeper than original design in right bank, was implemented to treat previous interbeds basalt flows at foundation area between the dam and the spillway gate structure.

2.4 *Itapebi*

Itabebi CFRD in Brazil (2002), 110 m high, 60,000 m^2 slab area, A/H^2 = 5.5, where after first filling impounding cracks were recorded in slab areas and leakage raised to 900 l/s. Treatment was carried out by dropping non cohesive sand soil from dam crest, similar as done in Ita dam. Excessive rockfill deformations caused deflections at slab top areas and no relation with foundations infiltration, according to further investigations.

2.5 *Tianshengqiao 1 (TSQ-1)*

TSQ-1 CFRD 178 m high, located in China, A/H^2 = 5.70, rockfill volume 17.7 million m^3 and 173,000 m^2 slab surface area, where cracks and concrete rupture were observed along the vertical joint at central part between slab 3 and 4, after impounding periods in 2003 and also 2004, consequently.

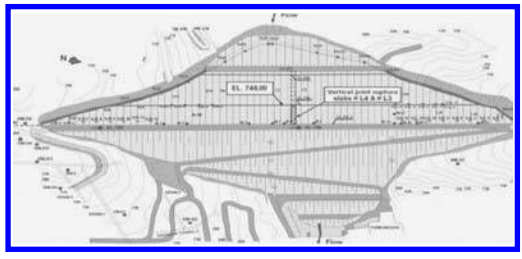

Figure 2. TSQ-1 face slab view and rupture along vertical joint, L4-L3 (three years after reservoir impounding).

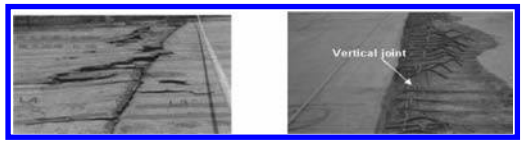

Figure 3. TSQ-1 Slabs #L4 & #L3—June 2003 (left)—Slab #L4 folded due compressive tension (right).

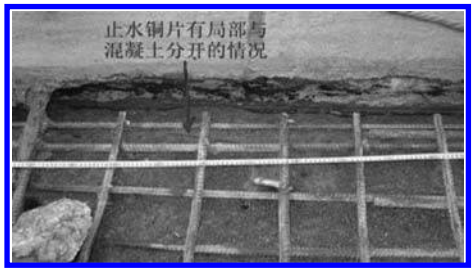

Figure 4. Spalling in face slab along the vertical joint in TSQ-1.

Spalling concrete parts and steel reinforcements were exposed along the vertical joint in a band of 1.0 m wide. Figure 3, shows #L4 slab steel reinforcement folded due slab tensions and rupture. In TSQ-1, concrete spalling at face slabs were caused by concentrated stress along vertical joints slabs L3 and L4. A maximum horizontal strain recorded was –912 μ (in strain gage SGH20) (where minus means compression) in June 2003 (Keming et al., 2007 and Freitas M., 2009).

2.6 *Sanbanxi*

The dam is located in China by 185.5 m high and 84,000 m^2 concrete face. The reservoir was filled in Jan. 2006 for the first time and then in process impounding dramatically until July 2007. Due to once-in-10 year flood, water level was increased to 5 m and abruptly seepage boosted from 113 l/s to 255 l/s. When the level declined, the seepage continued increasing to 303 l/s. On 7 August 2007 many instruments failure was observed in the right bank.

In Jan. 2008, the reservoir W.L. dropped down to dead operation level. So, underwater inspection detected damages in phase 1 and 2 construction joints about 184 m long at El. 385, including local concrete rupture, bent and exposed steel bars. In addition, broken vertical joint sealed with the Max. width of damaged panels closed to 4 m and Max. depth up to 41 cm. Repair works including remove of broken concrete, underwater placing polymer concrete, introducing joint bars and sealing vertical joints were implemented. Consequently, seepage was reduced about 100 l/s.

2.7 *Barra Grande*

Barra Grande CFRD, located in Brazil, 185 m high, A/H^2 around 3.0, 108,000 m^2 slab area, rockfill volume 11,850,000 m^3. During the reservoir impounding from July to September 2005, the W.L. achieved 163.0 m, on September 19, 2005 (91%, of the maximum normal Level); the leakage recorded was 220 l/s. On Sept. 22, the monitoring team observed a rupture along the center slabs #19 and #20 vertical Joint.

In addition, a gap (max. 12 cm) was found out beneath the slab ruptures areas, during a site inspection, Photo 6. This gap due the stiffness difference of the concrete slab and the rockfill, has been recorded in other CFRDs end of construction phase, such as: Xingó (145 m), Tianshengqiao 1 (178 m).

A local inspection carried out between El. 647.50 and 643.00 showed a vertical joint rupture between

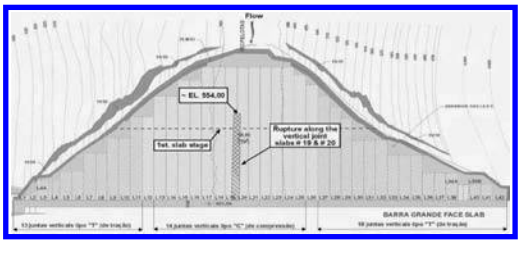

Figure 5. Barra Grande view and rupture along vertical joint.

Figure 6. Barra Grande—Sept. 2005.

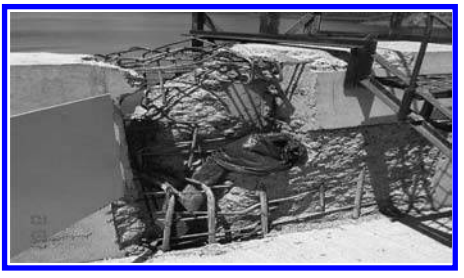

Figure 7. Parapet wall rupture in Barra Grande.

the slabs 19 and 20 at slab center part. Besides, parapet wall damages were observed at dam crest at same direction of slab 22. Additionally, the body–guard wall along the dam crest road was damaged due to compression efforts.

2.7.1 *Rehabilitation in Barra Grande*

The Barra Grande concrete repairing was performed in the following steps:

- Removal of the concrete damaged parts;
- Treatment of the vertical joint and bottom copper waterstop;
- Repairing reinforcement areas;
- Placement of additional anti spalling reinforcement;
- A additional slab, 0.30 m was concreted over the affected area, keeping the vertical joint along the vertical joint slabs No. 19 and 20;
- A flexible joint was placed along the joints 19 and 20 (above W.L.); and
- Additionally, a flexible joint was placed along all horizontal construction joint (2nd and 3rd face slab stages) at El. 640,0 m.

After the repairing works, reservoir impounding was restarted on Nov. 10, 2005, when the W.L. was at El. 630.1, and the leakage around 870 l/s. Additional remedial treatment by dumping silt sandy material have been promoted aiming to reduce and to keep the leakage around 600 liter/s.

In Feb. 2006, leakage records has increased and achieved 1,200 l/s (W.L. aprox. 631.2). In July 2009, leakage has ranged up 900–1,000 l/s, with the reservoir W.L. 635.0 to 647.0. No further treatments have been implemented, so far.

2.8 *Campos Novos*

Campos Novos CFRD, in Brazil, 202 m high, A/H^2 = 2.60, rockfill volume 12,000,000 m^3, 106,000 m^2 slab area. During the first reservoir impounding (Oct. 2005), the Water Level (W.L.) achieved 92% of maximum normal operation, when a rupture along the vertical joint between slabs #16 and #17, was observed at the center area

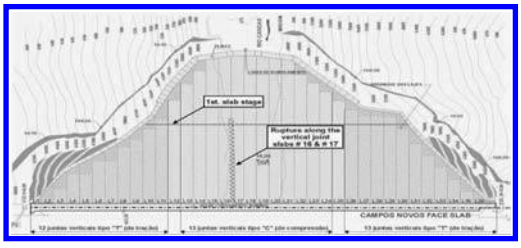

Figure 8. Campos Novos—Face slab general view and rupture along vertical joint (Ref. 8650/BP 30-A1-1004 modified- ENGEVIX).

Figure 9. Compose Novos Face slab general view after drawdown and upstream fill removed in July 2006.

of the concrete face. The rupture extended along all the vertical joint up to El. 660.0 (see Figure 3) and underwater down to El. 535 according to local inspection by the R.O.V. (Remote Operated Vehicle). In addition, horizontal cracks were observed at El. 656, along a construction joint between 2nd and 3rd slab stages. As recorded in Barra Grande CFRD, a gap was observed beneath the center part slabs (max. 4 cm) at Campos Novos CFRD.

2.8.1 *Rehabilitation in Campos Novos*

The repairing works have included:

- Drilling and grouting works to fulfill the gap beneath the slabs;
- Removing the broken concrete and the bottom copper waterstop and its replacement;
- Removing and replace the folded reinforcement parts;
- Placement of a Styrofoam 5 cm thick along the vertical joint between the #16 and #17 slabs, topped with a mastic joint and covered by a asphalt strip; and
- Building a new concrete slab, 30 cm thickness, over #16 and #17 affected slab area.

On Feb. 24, 2006 underwater remedial treatment was carried out by dropping silty soil over slabs #16 and #17 joint area. In June 2006, the reservoir was emptied completely through the diversion tunnels and it was possible implement a local face slab inspection and repairing works. In end of 2009, leakage records ranged from 900–1,000 l/s, with a dropping tendency.

2.9 *Mohale*

Mohale CFRD, in Lesotho, 145 m high, A/H^2 = 3.6, face slab area 87,000 m^2, rockfill volume: 7,000,000 m^3), during the impounding (March 2006), a rupture along the vertical joint at center part by compressive tension was recorded.

This cracking and rupture issues as occurred in Campos Novos and Barra Grande CFRDs, during impounding are quite similar as well.

Central compression joints current design concept, introducing flexible materials, has aiming mitigate high compression stresses along vertical joints.

However, monitored strain gauge in collapsed slabs (Mohale, Tianshengqiao 1) showed that flexion stresses should pay an much important effect more than a compressive rupture, in the related spalling rupture cases at center slabs.

2.10 *Zipingpu*

On May 12, 2008, Zipingpu dam, in China, 156 m in high, 127,000 m^2 face slab surface, suffered a strong earthquake with 8.0 Richter scale. As a case-study of CFRD performance during the earthquake and also evaluation of more vulnerable portion for rehabilitation works give valuable

Figure 10. Vertical joint rupture-reinforcement folded in Mohale.

Figure 11. Repair of the damage, longitudinal joint in Zipingpu.

experience. Cracks and rupture of concrete face slab, settlement of upper part rockfill, downstream slope movement, spalling of parapet wall, gaps between slab and rockfill, in added. Waterstop system was not seriously damaged and dam is still structurally stable and safe.

2.11 *Shuibuya*

The highest CFRD by the 233 m high, in China, slab face area of 120,000 m^2, A/H^2 = 2.30, was completed in 2007. The face slab was divided into 58 panels where the first stage face was 19 panels.

2.11.1 *Types of cracks in Shuibuya*

Cracks in first stage of concrete face detected in totally 255 pieces including, 195, 54 and 6 for group I, II and III respectively. Cracks grouping are as follows.

Detecting width and length of the cracks with JG4–10 reading microscope and steel rule, meanwhile inspecting the depth of the cracks with an impact echo method and coring was tested for typical cracks. For group 1 treatment was carried out using SG305-C1 fluid rubber, 10 cm wide and 1.0 mm thick was painted over the joint after surface grinding and cleaning; additionally painted a coat of PSI-200 cement base fluid after solidification. See Fig. 3 for treating technique for Grouping I joint.

For group 2 and 3 cracks treatment carried out by gouging according the specification and then burred 2 ply grouting pipes along the joint opening, and chemical grouting was carried out using

Table 1. Cracks grouping in Shuibuya.

Group	Crack width (C.W), mm	Crack deep (C.D), cm
I	<0.1	–
II	0.1 ≤ C.W < 0.3	≤30
III	0.3	>30

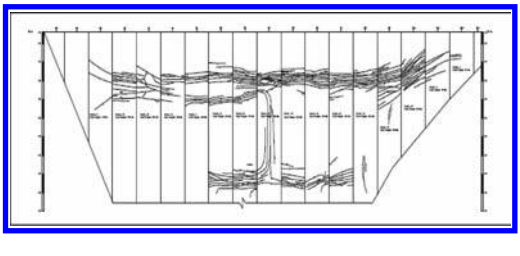

Figure 12. Freehand drawing for crack distribution in the first concrete face of Shuibuya dam.

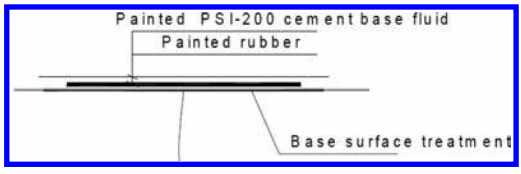

Figure 13. Treatment for group I cracks in Shuibuya.

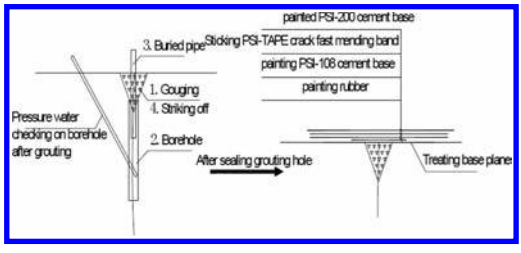

Figure 14. Treatment process for cracks group II & III.

pre-shrinkage cement mortar for striking off. The grouting agent was CW epoxy base fluid. The surface was grinded after having grouted and painted fluid rubber. Painted a 1.0 mm thick PSI-108 cement base fluid and stuck PSI-TAPE crack fast mended band along the openings.

2.12 *Mazar*

With the 166 m high, located in Ecuador, 45,000 m^2 surface of concrete slab, quite asymmetric abutments, in narrow valley (A/H^2 = 1.7), and 1.28 m Max. settlement in EOC.

In July 2008 after intense rainy period, increasing of rock foundation piezometer level in Zone 2A, low efficient drainage facilities and also unexpected issues probably have caused an uplift effect and moving up face slab below EL. 2030 m. In addition, slab edges upstream displacements have provoked high compression and flexion stress and spalling along plinth perimeter joint (bulging effect probably) (Costa G. Marcelo et al., 2009).

2.12.1 *Plinth rehabilitation in Mazar*

- Removing cover metallic bulb and fly ash containers
- Take out the top copper waterstop and removing of spalled concrete
- Concrete parts cutting 4 cm bellow the plinth reinforcement
- Anti spalling reinforcement bars (100 × 100 × 6) mm welding at the plinth reinforcement
- Placement new concrete (25 MPa at 28 days) and cutting works.

Figure 15. Upstream view of Mazar.

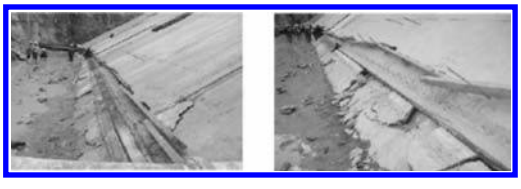

Figure 16. Perimeter joint view before and after top copper waterstop removing.

Figure 17. Plinth rebuilt concrete parts.

3 EXTRA HIGH CFRD NEW TRENDS

3. 1 *Bakun CFRD*

Bakun dam by the 205 m high, A/H^2 around 3, 127,000 m^2 face slab area, fill volume 16.7 million m^3, is located in the heart of jungle in Sarawak province in East-Malaysia, with an average annual rainfall 4500 mm. Face slabs 15 m width, was constructed in phase 1 with 18 slab panels and phase 2 with 50 slab panels.

3.2 *BAKUN dam design review*

Bakun dam was predicted in the risk and probably any failures, but according to the experiences learned from recent failures and also rehabilitation statement in a few CFRDs, where inappropriate vertical joint filling materials were applied and face construction revision was done.

The highlights points of Bakun´s original design and construction methods were reviewed as following:

- Increase the softwood filler thickness of the 6 central joints from 12 mm to 50 mm;
- Increase the thickness of the 10 central panels, 200 mm from EL. 125 m to EL. 229 m governed by the T = 500 + 3(229-H) mm;
- 0.4% reinforcement of the 10 central panels in both directions to be distributed 0.2% in each face;
- Eliminate the 100 mm V-notch at the 6 central slab in second phase;
- Delay placement of the central slab until the rate of settlement of the rockfill is less than 5 mm per month; and
- Increase the upstream zones 1A and 1B fills from EL. 100 m to EL. 125 m to cover about 50% of the dam height.

3.3 *Instrumentation results of Bakun at the EOC*

- Maximum settlement was recorded in EL. 110 m around the 1/3 H at DR. 350 m equal 227.7 cm;
- Maximum deformation recorded at elevation near the crest (78.1 mm) direction of deformation towards downstream and moving tendency to the river bed;
- Perimeter Joint—Up to 28th June 2009, the openness, shear and settlement values are smaller than the design value (60 mm);
- The analysis shows that most of the strain meters are under compressive state but very low rate, around 340 με; and
- The face slabs at DR + 543 and DR + 245 have a consistent deformation law as the lower part of the slab goes convex toward upstream while the upper part becomes concave toward downstream.

Performance of Bakun dam, based on Instrumentation values at the end of construction period,

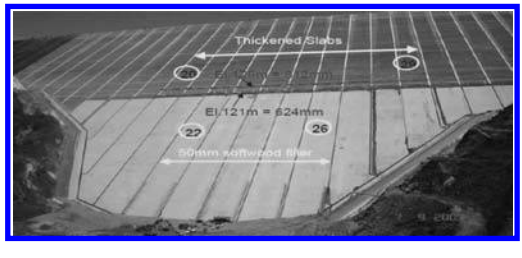

Figure 18. Bakun dam design review.

in generally all are adapted to measures and so far operating normally.

4 CONCLUSIONS

In this paper are discussed some cracks and failures cases recorded in several medium, high and also extra high CFRDs with rehabilitation as well as. Despite of dam steep slopes, empirical design criteria and current speed up construction methods, good performance behavior even after cracks and slab failures recorded and in very high seismic area as well as in close valley are examples of inherent CFRDs safety.

Moreover, the authors assume that even if any damage or failure is inevitable for extra high CFRD, simple and accessible repair procedures give a well gain of damage risk in construction and maintenance perspective for extra high CFRD.

Recent experience applied in Bakun dam, shown that no observed cracks in EOC by current design review which testify on the matter.

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Exposed geomembrane system at Messochora Concrete Face Rockfill Dam

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ABSTRACT: Messochora is a 150 m high Concrete Face Rockfill Dam in Greece, owned by Public Power Corporation (PPC). The bottom 52 m of the face slabs are covered by backfill. The dam, completed in 1996, was never impounded due to environmental reasons. In view of upcoming impoundment, PPC envisaged to rehabilitate the external waterstops that had deteriorated over the years, and to take preventive measures to avoid compression and tension cracking recently observed in some CFRDs of similar characteristics. A tensioned exposed geomembrane system will fully cover the face slabs in the lower 58 m, and the peripheral joint along the plinth up to the crest. A new external waterstop will waterproof the top 40 m of the vertical joints between slabs. Tension and compression joints will be treated differently, based on recent findings and recommendations of international experts. The paper describes design, components and installation steps performed.

1 BACKGROUND

1.1 *The dam*

Messochora is a 150 m high Concrete Face Rockfill Dam (CFRD) owned by Public Power Corporation (PPC). The dam, built in a narrow gorge of the Acheloos River in the north western part of Greece, has a 7.5 km power tunnel and a 2×80 megawatt power station. The reservoir has a total volume of 358,000,000 m^3, and a live storage volume of 228,000,000 m^3.

The geometry of the maximum cross-section of the dam and its material zones are shown in Figure 1. Construction of the embankment was completed between 1991 and 1995. The 23 slabs, poured after 6 months, are 15 m wide and up to 258 m long, have 1:1.4 inclination, and are designed with varying thickness in accordance with the formula t = 0,3 + 0,003H, where t is the perpendicular slab thickness in meters and H is the water head in meters below top water level (Thanopoulos 2006).

From elevation 625 to elevation 680, approximately one third of their height, the face slabs are covered by silty fill.

The peripheral joint between the plinth and the face slabs has a multiple line of waterproofing defence consisting of an embedded copper waterstop, of an embedded neoprene rod and joint filler, and of an external waterstop made by a 3 mm thick neoprene membrane sealing an Igas Black mastic filler (Fig. 2). Joints of similar type

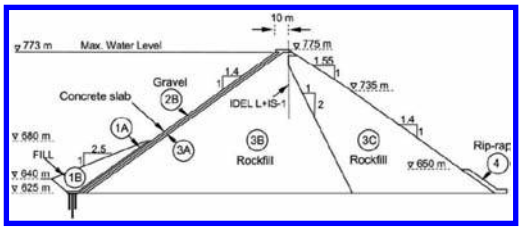

Figure 1. Maximum cross section of Messochora dam.

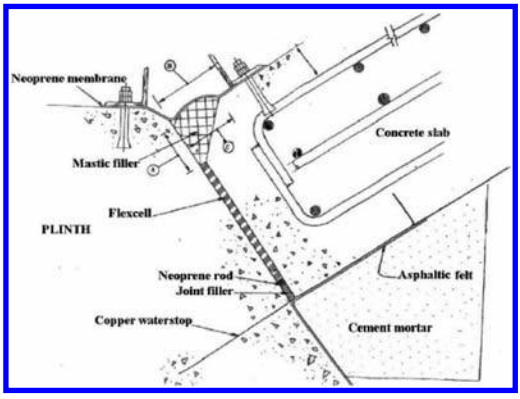

Figure 2. The peripheral joint with embedded copper waterstop, neoprene rod & mastic filler, and external waterstop.

were also installed between the slabs. The joints are designed to accommodate three directional movements of about 10 cm. Installation of the joint filling materials was done in 1996–97.

The neoprene membrane of the external waterstop is secured to the slabs along its perimeter by 65 × 65 × 6 mm L shaped stainless steel profiles, 3 m long, bolted to the concrete by 6 mm expanding anchors at 600 mm centres. A fillet of resin seals the external edges of the steel profile to the concrete.

1.2 *Post-construction period*

As discussed by Thanopoulos (2006) the opposition of environmentally active groups repeatedly postponed impounding of the reservoir for many years after the completion of the dam and appurtenant structures. The opposition mainly focused on the larger Acheloos River diversion scheme, but consequences concerned also Messochora HEP.

Some years after completion of the dam, in order to protect the external waterstop against environment aggression, mainly UV rays, an additional protection made by an angular steel element, 1 mm thick and approximately 60 cm wide, was installed on all joints (Fig. 3).

As impoundment kept being delayed, due to environmental aggression over the years several deterioration signs were observed that showed the need for repair. With special reference to the dam, repair of the concrete slab joints was an important issue, as they had been seriously affected. At many locations the protective steel plates had been lifted and torn by wind, exposing the neoprene membrane.

The exposed membrane was seriously damaged, presenting several fissures and cuts, so that it could

Figure 3. General view and detail of the steel protection over the external waterstops partially removed by wind.

Figure 4. Deteriorated external waterstop.

be manually torn and pulled off, and at several locations not providing any more the function of containing the mastic, as shown in Figure 4.

The external membrane waterstop on vertical and peripheral joints being no more totally watertight, it was deemed that and the entire waterstop system may not perform satisfactorily when the reservoir would finally be impounded, and movements expected to be up to 10 cm would occur.

1.3 *Problems of high CFRDs and remedial measures*

In view of upcoming impoundment of the dam, PPC at first envisaged to reintegrate the external waterstops exclusively in the sections where they had deteriorated, namely at the peripheral joint between the plinth and the face slabs, and at all vertical joints except 5 central compression joints that were designed without the mastic filler.

In the meantime some incidents had occurred in CFRDs having similar height and located in narrow valleys, where the compressive stresses that developed at the vertical joints and even inside the slabs caused rupture of the concrete and spalling, requiring extensive repair. Based on the similarities between Messochora dam and the dams where the incidents occurred, there was a concern that the settlements that would occur in Messochora embankment after impounding could cause the face slabs to deform under the pressure of water, and localized stresses to develop in the concrete, with opening of the already existing cracks and of new cracks, with possibly unacceptable water infiltration. More precisely, Messochora's valley shape ratio $A/H^2 \approx 2.2$ combined with hard limestone rockfill properties, of high void ration and anticipated grain breakage at filling of the reservoir were defavourable factors to be taken into account.

A number of solutions to prevent the risk of excessive seepage were examined by PPC, including

the application of a Poly Vinyl Chloride (PVC) sheet to the entire surface of the slabs at lower elevations. With the additional expertise and advice of an independent consultant the situation was thoroughly evaluated to ascertain the best measures to be adopted at the dam, also considering the remedial/preventive measures that were being undertaken or discussed by several experts in various parts of the world for the dams on which the incidents had occurred.

An analysis of the dam behaviour done with a 3D finite element tool simulating construction and impounding stages showed the development of important stresses in the slabs (Dakoulas et al. 2008).

The decision was finally taken to provide a full face waterproofing system to the face slabs in the lower part of the dam and to the peripheral joint up to crest, and to rehabilitate all the existing external waterstops, in a way as to avoid compression and tension cracking of the slabs.

2 DESIGN OF THE WATERPROOFING SYSTEM

2.1 *Design criteria*

The objectives were to provide long term imperviousness at the peripheral and vertical joints, and to avoid the compression cracking that had been observed basically in the central area, in the lower 40 to 45% of the dam height of the slabs, in those other dams sharing similar characteristics with Messochora. The similarities were considerable height (from 145 m up), relatively compressible fill (E~45 MPa), and narrow valley profile (ratio between slab area and the square of the height was less than 4).

For the vertical tension joints, a system that could prove long-term successful performance in similar conditions was required. PPC had direct satisfactory experience of such a system: in 1998, an external PVC waterstop had been installed on the vertical construction joints of Platanovryssi 95 m high Roller Compacted Concrete dam (Stefanakos & Dunstan 1999), and based on the successful performance the same external waterstop had been used four years later as underwater repair measure for a crack that opened when the dam was impounded, where the waterstop totally stopped the leak (Papadopoulos 2002). Since the same system had a successful precedent also of application on joints of CFRDs, as shown by its performance at Strawberry CFRD in USA (Scuero et al. 2003), it was deemed adequate for the vertical tension joints.

A further issue was how to treat the five compression joints, in consideration of the lessons learned with the recent incidents. It was decided to treat the two central compression joints in such a way as to avoid compression cracking.

To mitigate compression stresses, at the two central compression joints the slabs were to be cut vertically so as to introduce a 5 cm gap that would allow movements of the slabs occurring without causing cracking. To support the external waterstop over the cut, at first a filling material was considered. To be effective, the filling material had to be such as to allow placing it easily in the slot, with no risk of it flowing down the slope during installation and operation, and had to remain fluid and elastic over time, to ensure long lasting performance in as-installed conditions.

There was no proven evidence of a filling material compressible and elastic, that would not squeeze under the hydrostatic load and would be easy to place: if it were too soft and liquid it would not have been strong enough to take the load and it would have been very difficult to place because it would slide down along the slopes; on the contrary if it were easy to place because of low viscosity, then it would not have been that elastic. Some filling material that looked adequate had no documented experience, so here was no certainty that it would remain elastic over the years, and that after the first cycle of compression it would be able to go back to its original volume. In case the filler would not go back to its original volume, an empty gap would be created below the geomembrane of the waterstop, which could consequently intrude in the joint and be damaged. Furthermore, should the filling material become stiff over the time, it could puncture the geomembrane.

An approach similar to the one used by the external waterstops was finally adopted: instead than by a filler placed inside the gap, the support to the geomembrane was to be provided by a robust support layer placed externally, over the gap.

Another objective was to provide on the lower part of the dam, which would be subject to higher head and where cracks were already present, a water barrier capable of accepting considerable movements and bridge the further opening of the existing cracks and the onset of new cracks. The same exposed geomembrane system installed in 1997 on the Thissavros pressure tunnel, and that was providing very satisfactory performance, was selected. The decision on where to install the full face geomembrane system was selected in function of the foreseen load conditions and of the conditions of the face slabs. This resulted in a an exposed PVC geomembrane system covering the entire surface of the slabs, including the peripheral joint, from elevation 677 to elevation 735, and in a 15 m wide strip of exposed PVC geomembrane system following the peripheral joint at both abutments from elevation

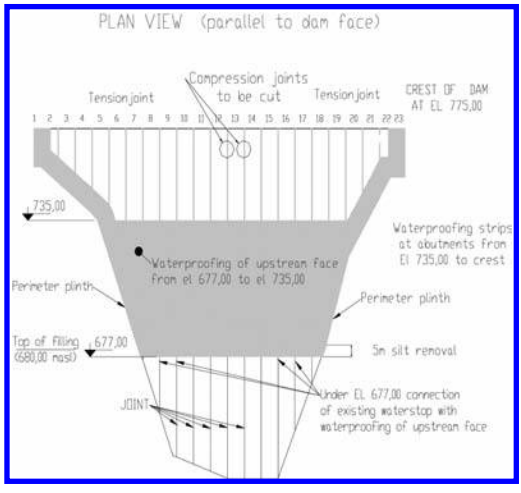

Figure 5. Arrangement of the new waterproofing system: full face waterproofing from elevation 677 to elevation 735, waterproofing of peripheral and vertical joints from elevation 735 to crest. Two central compression joints cut to avoid compression cracking.

735 to crest (elevation 775). The existing backfill was to be removed from elevation 680 down to elevation 675, to allow extending the exposed geomembrane system to elevation 677, and allow connecting the geomembrane waterproofing the face slabs to the existing external waterstops below elevation 677.

The new full face waterproofing system would waterproof cracks already present at some locations, compression or tension cracks that could develop during impoundment, and bridge the opening of the peripheral joint and of the two tension joints at the abutments.

2.2 *The external waterstop on tension joints*

The tension joints are waterproofed with Carpi patented external waterstop, based on the concept of installing a wide (in the order of 50 cm) strip of robust flexible PVC material, and anchor it at the periphery so that the free space for elongation is much higher than in a conventional embedded PVC waterstops. With elongation exceeding 230% and free elongation over 50 cm, the external waterstop will be capable of accommodating movements of the joints much larger than those anticipated.

The PVC liner is supported over the joint by flexible sacrificial layers designed in function of local conditions. At Messochora, the already existing external waterstop was to be taken into account. Based on the conditions of the existing components and on evaluation of the related benefits and disadvantages, the decision was taken to completely remove the protective steel plates and the L shaped anchorage profiles, and to remove or leave in place the existing neoprene membrane and mastic filler depending on the extent of the deterioration. The layering of the new waterstop, proceeding from the face of the slabs upwards is (Fig. 6).

- One non-woven needle punched anti-puncture geotextile 2000 g/m^2
- One sacrificial layer of PVC geocomposite, consisting of a 2.5 mm thick PVC geomembrane heat-bonded during fabrication to a 500 g/m^2 geotextile
- One waterproofing PVC geocomposite, the same used for the sacrificial layer.

The anti-puncture and sacrificial layers are mechanically anchored at the sides; the waterproofing geocomposite is anchored along the entire perimeter with a watertight perimeter seal of the tie-down type, made by compressing the PVC geocomposite on the concrete with stainless steel batten strips, with regularising resin and rubber gaskets and splice plates to ensure even compression. The same seal had been used at Platanovryssi dam and had been previously tested at 240 m water head.

The external waterstops are placed from elevation 735 to crest, on all tension joints and on compression joints 10–11, 11–12 and 14–15.

2.3 *The external waterstop on compression joints*

The two central compression joints are treated to allow movements of the slabs avoiding compression, sliding of adjacent slabs over each other, and spalling of the concrete. After removal of the existing deteriorated external waterstop, two parallel vertical cuts are made along the joints at approximately 5 cm distance, centred on the joint and having a regulated depth so as to come as close as possible to the copper waterstop in the slabs without damaging it. The concrete is then removed to leave an approximately 5 cm wide groove in each joint.

To support the external waterstop over the groove, a 50 cm wide strip of conveyor belt material is placed centred on the groove and mechanically anchored on both sides to the slabs. The layers on top of the conveyor belt are the same of the tension joints (Fig. 7).

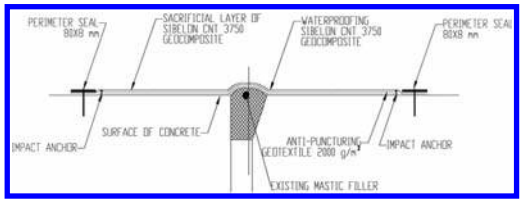

Figure 6. Patented external waterstop on tension joints.

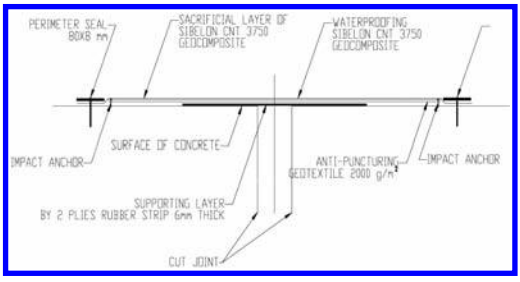

Figure 7. Patented external waterstop on the two central compression joints.

2.4 *The full face exposed geomembrane system*

The waterproofing geocomposite, a 2.5 mm thick PVC geomembrane heat-bonded during fabrication to a 500 g/m^2 geotextile, is anchored to the slabs by a mechanical system amply discussed in international literature, among which Bulletin 135 published by the International Commission on Large Dams (ICOLD 2010).

The mechanical anchorage system is composed by a vertical face anchorage and by a peripheral anchorage. The face anchorage consists in Carpi patented tensioning profiles, placed at 5.7 m spacing, which allows a corresponding free width of elongation for the PVC geocomposite that is much larger than the one foreseen for the external waterstops. The tensioning profiles are waterproofed with a strip of PVC geomembrane 2.5 mm thick and identical to the geomembrane that composes the waterproofing geocomposite, but without backing geotextile, watertight welded upon the geocomposite. The tensioning anchorage will avoid formation of slack areas and folds, in line with the recommendations of Bulletin 135.

At all vertical joints and at peripheral joints, the existing neoprene membrane is removed, the existing mastic filler is cut flush to the surface, and the part remaining underneath is left in place, to act as a support to the geomembrane. The support layers are the same adopted for the external waterstops.

On the surface of the slabs, a drainage geonet is placed under the geocomposite, serving also as anti-puncture protection. Where the waterproofing system crosses the joints, the geonet is placed between the sacrifice geocomposite and the waterproofing geocomposite. The tensioning profiles will act as vertical drainage channels conveying the water to the drainage geonet. Drained water will discharge into the rockfill.

At top (elevation 735), at bottom (elevation 677), and around the 15 m wide strips at the abutments, the geocomposite is fastened by the same watertight seal used for the external waterstops. An extra strip of geocomposite will be left extending outside the top seal at elevation 735 and at the peripheral joint, in order to allow connecting the geocomposite to a new geocomposite which may be added in the future to cover the whole face in case the top slabs crack.

At the base of the parapet wall, the waterproofing geocomposite is fastened by a 50 × 3 mm flat stainless steel batten strip.

2.5 *Treatment of joints at elevation 677*

At elevation 677 the joints must be treated to minimize water infiltration at the joints from the section below, where there is no liner.

At tension joints the L profiles are removed, the neoprene membrane and the mastic filler are cut with an appropriate inclination to make a tapered connection on top of which a strip of PVC geomembrane is installed. The PVC geomembrane strip is watertight welded to the PVC geocomposite waterproofing the slabs, and watertight glued to the neoprene membrane of the joint. The L profiles are then reassembled anchoring both the neoprene membrane and the PVC geomembrane.

At the five compression joints, at the elevation where the bottom seal is made, a hole is made in the concrete, centred on the joint and down to the maximum depth such as to not intercept the existing waterstop. The hole is filled with a polyurethane based resin and the bottom seal is installed on top of the filled hole, so that watertight connection is achieved between the new external waterstop and the existing internal waterstop.

2.6 *Monitoring system*

To monitor future performance of the waterproofing liner, an optical fibre cable system is installed under the PVC geocomposite, along the perimeter of the full face lined section. The optical fibre cable system, which is based on the capability of optical fibre cables to detect changes in temperature due to water infiltration, is the same adopted at Brandbäch and Waldeck dams in Germany, at Kadamparai dam in India, at Mittlerer Isar canal in Germany.

Additionally a monitoring system of the performance of the slabs is to be installed, including three lines of in-place inclinometers along the vertical joints and sets of 3-axial extensometers allowing the measurements of the movements at three elevations along the peripheral joint and on centrally located slab-to-slab joints.

The whole monitoring system, although watertight, is designed to be covered by the geomembrane system.

3 INSTALLATION

3.1 *Central compression joints*

Waterproofing works started in November 2009. The cut at the two central compression joints was the first task. The L profiles were removed and the existing external waterstop was cut flush to the concrete surface. The two parallel cuts were then made with a circular saw. To assure that the depth of cutting would not reach and damage the internal copper water stop, a drilling device incorporating an automatic stop system was set to precede the circular saw. The drilling device executed probing to ascertain the depth of the copper plate. After the depth of the copper plate had been detected, the circular saw was calibrated for a depth 5 cm less than the detected depth.

Cutting was executed by a specialty contractor working from a travelling platform secured at the crest of the dam. Operations were complicated by the fact that the joints were not strictly vertical, and a few days more than the foreseen 30 days was required to complete the two cuts. The concrete between the two cuts was removed with an electric hammer down to a depth of about 5 cm above the copper waterstop.

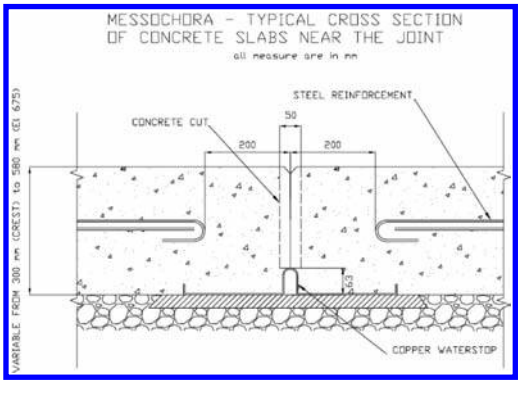

Figure 8. Detail of joint cutting at the two central compression joints.

Figure 9. View of the dam during cutting of the two central compression joints.

3.2 *Installation of temporary walkway at elevation 706*

Due to the dimension of the dam, for the installation of the waterproofing system it was decided to divide the upstream face in three horizontal sections, the bottom one from elevation 677 to elevation 706, the intermediate one from elevation 706 to elevation 735, and the upper one from elevation 735 to the crest. Installation started at the lower section (full face system).

To have a supporting deck for workers and for storage of materials to be installed on the lower section, a temporary walkway secured to the face slabs at elevation 706. The temporary walkway is a steel structure made by steel beams fixed to the concrete slabs and supporting a flat deck made by steel plates. When the work on the lower section is completed, the structure will be dismantled, lifted and placed again at elevation 735 to allow working on the intermediate section. The upper section will be waterproofed working directly from the crest of the dam.

As shown in Figure 10, personnel and materials are transported from the crest of the dam down to the temporary walkway by means of an electrically powered trolley secured to the parapet wall. The trolley travels up and down sliding on steel rails secured to the concrete slabs. Four suspended platforms anchored to steel structure of the walkway are used access the surface of the dam at various levels where waterproofing works are performed.

3.3 *Installation of full face system in the lower section*

To allow extending the full face system down to elevation 677, the existing fill was removed from elevation 680 down to elevation 675. After installation of the geomembrane system is completed, the fill will be restored from elevation 675 up to

Figure 10. The walkway at elevation 706 and the yellow trolley for transport of personnel and materials.

elevation 680. To avoid puncturing the geomembrane by the backfilling, a protection geotextile will be placed on top of the new geomembrane from elevation 677 to elevation 680.

Waterproofing works started on December 14 with installation of the first component of the vertical face anchorage system, a U shaped profile that was secured to the face slabs with chemical phials.

After about one month of interruption due to holidays/delays caused by installation of the instrumentation for the dam/exceptionally bad weather, works were resumed with preparation of the vertical joints and relevant support layers as applicable. The drainage geonet was then placed on the free space between adjacent vertical profiles, as shown in Figure 11.

Concurrent with installation of the U profiles and geonet, placement of the optical fibre cable started. The PVC geocomposite rolls were then deployed from the temporary walkway down to the bottom, and secured by fastening the second, omega shaped component of the vertical tensioning assembly. To avoid water infiltration at the threaded anchor bars that cross the PVC geocomposite, the omega shaped profiles were then covered with PVC geomembrane strips as discussed in the section on design. Figure 12 shows installation of the PVC geocomposite sheets.

Figure 11. Placement of U profiles and drainage geonet. At left, the support layers on the peripheral joint can be seen.

Figure 12. Placement of PVC geocomposite sheets over the drainage geonet.

At the end of February 2010 works were temporarily interrupted due to a decision of the Council of State that upheld the appeal presented by environmentalists against any works to be carried out in the Acheloos River.

When works were interrupted, the face waterproofing system had practically been completed on about one third of the lower section, and was about half-way on another third. Before the crews left the site precautions were taken to provide a temporary sealing of the already installed sections that would avoid possibility of loosening the system by wilful action.

PPC has taken action to reassume environmental assessment for the project, as it is independent from the larger Acheloos water management scheme.

4 CONCLUSIONS

The waterproofing geomembrane system of Messochora dam has been designed based on the most recent international experience and on the best state-of-the-art of geomembrane technologies.

If the project is allowed to proceed up to completion, safe operation of the dam under any service conditions will be granted.

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Actuaciones de mantenimiento y rehabilitación en la presa de "El Águeda". (Salamanca)

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RESUMEN: La presa de El Águeda tuvo sus comienzos en los años veinte del siglo pasado. Han sido diversas las actuaciones llevadas a cabo tanto en la presa, como en los aliviaderos, para adecuarla a las necesidades de explotación y seguridad. Para ello se han realizado tratamientos de mejora y consolidación en los dos aliviaderos, tanto el central como el de margen izquierda, se ha tratado el paramento de aguas arriba y el de aguas abajo, se ha demolido una galería en el cuerpo de presa para inspección y recogida de drenajes, se han sustituido las compuertas Taintor instalando ataguías en el aliviadero central, y se han sustituido los desagües fondo empleando para ello escudos subacuáticos. De todo ello exponemos a continuación las soluciones adoptadas, tras valorar las más adecuadas en cada caso, así como los procesos de ejecución para llevarlo a cabo.

ABSTRACT: The dam of El Águeda had its beginnings in the twenties of last century. There has been various actions carried out, both in the dam as in the spillways, to adapt the needs of exploitation and security. To this effect, improvement and consolidation treatments has been taken in the two spillways, central and left margin; upstream and downstream faces have been rehabilitated; a new gallery has been built, by demolition, in the dam body, aimed to inspection and collection of seepages; the Taintor gates in the central spillway have been replaced, installing cofferdams upstream and, finally, the bottom outlet valves have also been replaced, using underwater shields. The solutions adopted, after considering the most appropriates in each case, are presented together with the implementation processes.

1 ANTECEDENTES

La presa de El Águeda está situada a unos 10 Km. aguas arriba del municipio de Ciudad Rodrigo (Salamanca), y embalsa agua del río del mismo nombre.

Es una obra de fábrica de planta curva, datando de los años veinte del siglo pasado.

En sus inicios la presa estaba constituida por un dique que cerraba el cauce dejando la función de aliviadero a un canal en margen izquierda. El desbordamiento de agua sobre la presa llevó a habilitar un portillo central sobre el dique al que posteriormente se dotó de compuertas quedando la presa con el aspecto que ha conservado hasta nuestros días.

El aliviadero lateral estaba excavado en roca. Los diferentes vertidos atacaron la sección con gran pérdida de material y amenazando con destruir el canal de alimentación a la zona regable y el caudal de mantenimiento del río.

En el año 1991 se comenzó la actuación sobre el aliviadero lateral consistente en ejecutar la solera del canal de descarga y gunitar las paredes laterales con anclaje de la roca, previa colocación de un mallazo de protección.

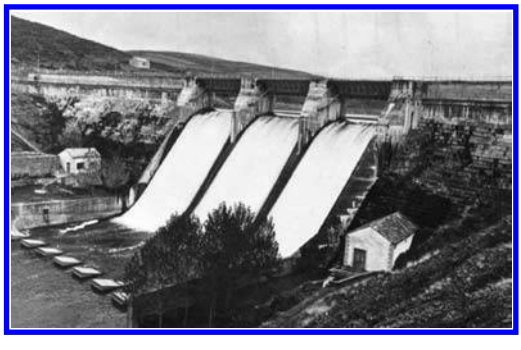

Figura 1. Presa de El Águeda.

El escalonado del paramento aguas abajo había sufrido un deterioro en su geometría rehabilitándose con mortero de cemento con resina epoxi, consiguiendo mantener la geometría del paramento hasta el momento presente.

En esta primera fase se efectuó una reparación del cuenco amortiguador que había sufrido deterioros con los sucesivos vertidos.

La segunda actuación se generó a instancia de nuestro querido Guillermo Gómez Láa. En

una de las visitas a las actuaciones reseñadas con anterioridad, se planteó la conveniencia de demoler una galería en el cuerpo de presa para ahondar en el conocimiento de la misma.

Ello llevó a la segunda actuación consistente en una regeneración del paramento aguas arriba de la presa, tratamiento de la parte superior aguas abajo, saneamiento del vertedero y tratamiento con mortero de resinas y ejecución de galería en el cuerpo de presa.

El efecto de la tipología y el paso del tiempo produjeron un deterioro agudo empezando las actuaciones de rehabilitación en 1991.

El objeto de la ponencia es la exposición de estas actuaciones efectuadas en la presa.

- La primera actuación consistió en la rehabilitación del aliviadero lateral de la presa que había sufrido la erosión de los sucesivos vertidos. Se completo la actuación con una regeneración del paramento agua abajo.
- En una segunda actuación se regeneró el paramento aguas arriba, el paramento del aliviadero central y se ejecuto una galería de observación y drenaje. La instalación de un sistema de control e instrumentación propició el descubrimiento de un fenómeno de despegue del cimiento que llevó a una campaña de inyección de consolidación y cosido. Se finalizó la actuación con una adecuación del entorno de marcado carácter medioambiental.
- La existencia de unas compuertas isostáticas de celosía, sin ataguías y la obstrucción de los conductos de los desagües de fondo con la inutilidad de los mecanismos llevó a la última actuación llevada a cabo hasta el momento, aplicando los conocimientos de otras actuaciones, consistente en la instalación de ataguías metálicas extraíbles mediante vaineros, en los tres vanos del aliviadero, reemplazando las compuertas existentes por otras con sistema de viga cajón mucho más resistente y accionamiento mediante cilindros hidráulicos. Instalación de un sistema de elevación, transporte y colocación de las ataguías mediante doble polipasto móvil sobre viga carril y viga tenaza. Colocación de escudos en las embocaduras de los conductos de los desagües de fondo con el empleo de buzos. La costosa preparación necesaria, la realización del dragado de fangos y las demoliciones precisas en hormigón; todo ello mediante técnicas subacuáticas, dado que las embocaduras estaban en contacto con la cimentación de la presa y en época de la construcción, como se dedujo a posteriori, se cimentó fuertemente para permitir la circulación a través de los conductos.
- La ejecución de las demoliciones necesarias en el interior de cámaras y conductos para la extracción de compuertas obsoletas y nueva instalación de compuertas y blindajes. Proceso de hormigonado e inyección de los mismos.

De todo ello se expondrán los condicionantes de la presa para determinar las soluciones adoptadas, el proceso de elección de los equipos, los sistemas de trabajo empleados y el resultado final de los trabajos realizados.

2 REHABILITACIÓN DEL ALIVIADERO LATERAL

El aliviadero de la margen izquierda, era el único del que disponía la presa inicialmente, ya que el central se construyó años después de la puesta en explotación de la presa para aumentar la capacidad de desagüe, ya que el caudal punta de la presa se estimó en el año 1934 en 1.500 m^3/sg.

El aliviadero lateral estaba excavado directamente en la roca, sin ningún tipo de revestimiento, con una longitud de algo más de 100 m. hasta desaguar al río, con taludes muy fuertes y cauce muy irregular, con calados y soleras que van respectivamente de 4,00 m. y 10,00 m. en origen y de 10,00 m. y 15,00 m. en el tramo final.

El paso del agua y la meteorización provocaban, desprendimiento de rocas y un gran deterioro general tanto de las soleras como de los cajeros del aliviadero, llegando a peligrar la estabilidad del cajero del lado izquierdo que soporta la ladera de la montaña.

La solución adoptada, consistió en dotar al aliviadero de una solera de hormigón armado, con un perfil mucho más uniforme e hidráulico para desaguar el agua. Los cajeros se trataron con un bulonado mediante barras de acero anclado a la roca, al que se fijaron mallazos metálicos, sobre los que se ejecuto un proyectado de hormigón. La parte superior del cajero lateral izquierdo, en su parte más alta, se dejo solamente con el bulonado, ya que se consideró que por estética e integración medioambiental, y puesto que el agua no llegaba hasta esa altura, se podía prescindir del gunitado.

Figura 2. Aliviadero lateral izquierdo.

3 TRATAMIENTO DEL PARAMENTO DE AGUAS ARRIBA, DEL VERTEDERO CENTRAL, EJECUCIÓN DE GALERÍA, E INYECCIONES

La patología del hormigón de la presa, hemos de decir que se corresponde con las condiciones del medio y las características constructivas empleadas en su construcción así como la deficiente calidad de los materiales propios de la época. Todo ello se manifiesta en un envejecimiento progresivo del hormigón que muestra una fuerte degradación superficial, debida a fenómenos de meteorización, erosión, sobre todo por efecto del agua y acciones de hielo y deshielo. Manifiesta discontinuidades estructurales que se exteriorizan en juntas estructurales, debido principalmente a que no tiene juntas de construcción.

La acción combinada de todas estas acciones acelera el deterioro de la estructura de hormigón, haciéndose las degradaciones cada día más profundas, ocasionando grandes coqueras que es preciso tratar para evitar que el proceso se incremente y ponga en peligro la integridad de la presa.

3.1 *Tratamiento del paramento de aguas arriba*

El paramento de aguas arriba presentaba en la zona superior de la misma, en los dos tercios superiores, un deterioro importante que se manifestaba en tongadas más o menos horizontales de aproximadamente 50 cms. de altura, en los que en la junta de cada tongada de hormigonado se observaba un fuerte deterioro y una importante pérdida de material, en torno a los 10 cms de profundidad llegando en algunos casos a los 15 cms, y más.

La solución a adoptar debía conseguir, por una parte resolver el problema de las juntas, por otro obtener un revestimiento con resistencia suficiente tanto mecánica, para mantenerse firmemente unida al paramento viejo, a la vez que flexible para adaptarse sin fisurarse a los movimientos propios de la estructura, como a los agentes externos y medioambientales a los que iba a estar expuesta.

Se proyectó un tratamiento que consistía, en un saneo previo mediante empleo de martillos picadores y limpieza con bomba de agua de alta presión, hasta obtener una superficie adecuada para el nuevo tratamiento. Tratamiento puntual de las juntas más significativas mediante puente con lámina impermeable de polietileno clorosulfonado tipo Hypalón unida a los bordes de la junta con resina de epoxi.

El tratamiento propiamente dicho, se realizó con un proyectado de hormigón de 300 Kg/cm^2 de resistencia mediante el empleo de máquinas de proyectar Alivas. Al hormigón así proyectado se le adicionaban fibras de acero tipo Drámix ZP 30/35

Figura 3. Proyectado y junta del paramento de aguas arriba.

con una dosificación de 40 Kg/m^3 de hormigón. Con estas fibras se conseguía el objetivo de que el hormigón resultante fuera muy resistente, pero que a la vez fuera mucho más flexible que un armado convencional. Para garantizar un buen agarre al paramento de la presa, además de la propia adherencia del producto, previamente al proyectado, se colocaba, aproximadamente, un anclaje por cada metro cuadrado de paramento de 40 cms. de profundidad con varilla roscada de Ø 20 mm. fijadas con anclaje químico. Se realizaron ensayos normalizados de resistencia a compresión in-situ por medio de arrancamiento (s/Norma UNE 83.604/94) de rotura por tracción de pernos, una vez finalizado el tratamiento, obteniendo unos resultados de rotura por encima de la resistencia exigida. Se realizaron siete ensayos a siete días, en los que se obtuvieron los siguientes resultados: 23, 23, y 28 N/mm^2, en tres casos se rompió el perno y en uno sin llegar a romper el perno se salía de rango es decir por encima de 33 N/mm^2. Este tratamiento se realizo hace 15 años y mantiene una conservación muy buena, y no ha manifestado ningún defecto.

3.2 *Tratamiento del paramento del vertedero*

El paramento del vertedero de acuerdo con lo mencionado en el estado general del hormigón de la presa, presentaba un aspecto deslavado por el paso del agua, con pérdidas puntuales de material y coqueras en algunos puntos y algunas juntas muy marcadas.

El tratamiento que se aplicó, consistió en lavado general con chorro de agua a alta presión y puntualmente en zonas con mayores adherencias con chorro de arena. En las zonas del paramento de coqueras o zonas de material desprendidas se regeneró con un mortero de epoxi-cemento con el espesor necesario.

Como tratamiento general a todo el paramento del vertedero, se aplicó un mortero de resinas sintéticas con un espesor de 1,5 mm. al que se superpuso

Figura 4. Tratamiento del aliviadero del vertedero central.

una malla de fibra de vidrio con una segunda capa de mortero de resinas epoxi de 2,5 mm. Finalmente como protección del tratamiento y acabado se aplicó una pintura de resinas sintéticas.

3.3 *Ejecución de demolición de galería en el cuerpo de presa*

La presa disponía de una galería situada a la cota 609,00 pero muy próxima al paramento de aguas abajo que sirve de paso de un lado a otro del bajo el aliviadero, ya que las entradas están junto a los muros del aliviadero y de acceso a las cámaras de los desagües de fondo. Desde esta galería se habían ejecutado taladros para controlar con manómetros las subpresiones, aunque en puntos bastante alejados del paramento de aguas arriba.

Se proyectó una galería que recorriera la presa de lado a lado a siete metros del paramento de aguas arriba, aproximadamente a la misma cota que la galería exterior, llegando al contacto con el terreno en ambos estribos y pasando por las dos cámaras de los desagües de fondo y uniéndolas entre ellas.

La galería se ejecutó con una sección de 1,68 m. de anchura y 2,18 m. de altura en el hormigón del cuerpo de presa, con una longitud de 103,53 m. La nueva galería sirve también de acceso a dos nichos, situados uno en cada margen de la presa, de 3,00 m. de longitud y 2,00 m. de anchura que perpendiculares a la galería, se acercan al paramento de aguas arriba quedando a tan sólo 4,00 m. de dicho paramento, y que alojan dos péndulos directos.

El objeto de ésta galería es servir de acceso a los camerines de los péndulos, drenar la presa y controlar las subpresiones y las filtraciones cerca del paramento de aguas arriba, y servir de punto de inspección y control del cuerpo de presa, todos estos extremos de gran relevancia para realizar la explotación y conservación de la presa.

Para la ejecución de la demolición, trabajo muy costoso y difícil, se empleo fundamentalmente

Figura 5. Demolición de la galería.

quebrantador hidráulico tipo darda, combinado con el trabajo de martillos perforadores y martillos picadores neumáticos. La calidad del hormigón extraído, fue bastante irregular en su granulometría, podríamos catalogarlo de ciclópeo, con bastantes huecos pero no carente de resistencia, lo que supuso un inconveniente al realizar la demolición ya que bajaba notablemente el rendimiento de los equipos.

Una vez realizada la galería, se construyo una solera de hormigón armado en la anchura de galería con una canaleta de 50 cm. y 5 cm. de profundidad cubierta con un trámex metálico que sirve de canalización para la recogida de las filtraciones.

La ejecución de esta galería y de los péndulos permitió comprobar, que la presa sufría unas importantes subpresiones, y posible efecto de despegue del terreno, que motivó la necesidad de realizar una campaña de inyección de consolidación y cosido.

3.4 *Ejecución de inyecciones*

Durante las operaciones preparatorias para la instalación de uno de los péndulos invertidos de la presa, al realizar la perforación del taladro necesario con un diámetro de 300 mm., se observó que el contacto del hormigón del cuerpo de presa con el terreno, estaba muy disgregado, meteorizado, con muy poca cohesión y adherencia entre ambos. Esto manifestaba, un fuerte empuje debido a las subpresiones, con un fenómeno de despegue del cuerpo de presa del terreno.

Para solucionar este problema, se planteó la realización de una campaña de inyecciones de lechada de cemento con el objeto de consolidar la cimentación y coser el cuerpo de presa al terreno mejorando la adherencia entre ambos y eliminando gran parte de las subpresiones, a la vez que consolidando el cuerpo de presa en el contacto con la cimentación.

Se realizaron taladros desde el interior de ambas galerías, en secciones separadas aproximadamente 4 metros. En cada sección se ejecutaron dos taladros en la galería cercana al paramento de aguas arriba, uno vertical y otro con inclinación (aproximadamente 30°) hacia aguas arriba, y tres en la que está cercana al paramento de aguas abajo, uno vertical y dos con inclinaciones similares opuestas hacia aguas arriba y hacia aguas abajo. Se ejecutaron con sonda de perforación mediante rotación, hasta profundizar 5 metros en el terreno.

Realizada la inyección desde las galerías, se ejecutó una pantalla de inyección mediante taladros hechos desde coronación, a 1,50 m. del paramento de aguas arriba, hasta penetrar 8 metros en el terreno. Con esta pantalla de inyección conseguíamos, por una parte consolidar e impermeabilizar el cuerpo de presa en un punto muy cercano al paramento de aguas arriba. Por otra crear una barrera de impermeabilización y consolidación en el contacto con el terreno igualmente en un punto muy cercano al paramento de aguas arriba, con lo que se redujeron notablemente, las filtraciones y las subpresiones en la presa.

Las admisiones en los distintos taladros fueron variables con admisiones que fueron desde los 200 Kg de cemento, hasta más de 22 Tn en el que más inyección admitió. En total se inyectaron 820 Tn de cemento.

4 SUSTITUCIÓN DE COMPUERTAS TAINTOR Y DESAGÜES DE FONDO

4.1 *Sustitución de compuertas Taintor e instalación de nuevas ataguías*

El aliviadero central, dispone de tres vanos en los que se encontraban instaladas sendas compuertas de 15,00 m. de longitud y 4,75 m. de altura. Éstas compuertas estaban formadas por una chapa forro y una estructura tipo celosía formada por barras y nudos de tipo isostático, con un dimensionamiento muy estricto para los esfuerzos que debían soportar. Todo ello unido al tiempo que llevaban en servicio, la corrosión de los materiales, el estado de las soldaduras y los distintos repartos de tensiones en las barras, planteaba una incertidumbre sobre su resistencia, de muy difícil valoración.

Intentar resolverlo reforzando la estructura existente, podía conducir a la aparición de tensiones motivadas por las soldaduras, lo que aumentaba todavía más la incertidumbre.

Por otro lado el sistema de actuación de las compuertas era mediante motores eléctricos que transmitían el movimiento a través de piñones de acero a unas cadenas que tiraban de las compuertas para elevarlas. Estas cadenas tiraban en ambos lados de las compuertas, pero la transmisión del lado en el que estaba el motor al otro lado de la compuerta, se realizaba mediante barras de acero y piñones cónicos, que debido a la torsión de las barras y las holguras en los piñones, producía desfases entre los dos lados de las compuertas y favorecía un posible acodalamiento.

Por todo ello se optó por la sustitución de las compuertas en su totalidad, dotando además a los aliviaderos de un sistema de ataguías móviles, que permitiera poder cerrar, alternativamente, uno de los vanos del aliviadero. Para el movimiento y elevación de las ataguías se proyecto un doble polipasto monorraíl, de 8 Tn. cada uno, con una pilona metálica situada en cada una de las cuatro pilas del aliviadero, que permitiría tanto desplazar cada una de las ataguías de un vano a otro como de elevarlas y bajarlas a su lugar de empleo.

Se proyectó la sustitución de las compuertas existentes por otras, de las mismas dimensiones, formadas por chapa forro y viga cajón, que proporciona una mayor inercia y resistencia a los esfuerzos, mejorando la distribución de los mismos.

En cuanto a las ataguías se decidió construirlas en tres partes de 15,00 m. de longitud y 1,85 m. de altura, lo que supone una altura total de cierre entre las tres de 5,55 m. El motivo de hacer tres ataguías, era que puesto que se dotaba a las ataguías de un elemento de elevación y transporte, para conseguir una total autonomía en su empleo, en vez de disponer un parque de ataguías en el exterior de la presa, para almacenarlas cuando no se estén en uso, se pensó que cada una de las tres ataguías quedaría colgada en cada uno de los tres vanos del aliviadero. De este modo quedan siempre a disposición para ser usadas con el simple empleo del monorraíl y los polipastos instalados. Quedaba todavía, otro problema por resolver para la instalación de los vaineros para la colocación de las ataguías. Aguas arriba de los viaductos que pasan sobre los aliviaderos, las pilas tajamares no tenían sección suficiente para instalar los citados vaineros. Los viaductos sobre los aliviaderos, estaban construidos con tres vigas en T de hormigón armado de 1,50 m. de canto, con alas de 50 cms. a cada lado. Se pensó que si se pudiera cortar el ala a dos de estas vigas, quedaría un metro de hueco entre ellas por el que poder introducir las ataguías.

Tras verificar los cálculos de las vigas en T, ésta fue la solución que se adoptó. Para realizar el corte de las alas de las vigas se empleo disco de diamante, con lo que se conseguía un corte perfecto, y no se

dañaba las vigas existentes. Se proyecto un marco metálico perimetral al hueco ejecutado, que serviría para apoyo de un trames para grandes cargas, que cubriría el hueco de las ataguías cuando éstas no estén en uso.

Se programaron los trabajos de tal manera que no interfieran en la normal explotación de la presa. Por ello los trabajos se realizaron ejecutando cada uno de los vanos del aliviadero de uno en uno, no comenzando los trabajos en otro vano hasta tener la compuerta instalada totalmente instalada y en funcionamiento en el anterior.

Primeramente, se instalarían los vaineros y la viga de cierre de las ataguías, y con las ataguías colocadas en un vano, se procedería al desmontaje e instalación de la nueva compuerta Taintor.

Para realizar los trabajos, tanto en las ataguías como en las compuertas con seguridad, fue necesario preparar una plataforma de trabajo al nivel de la viga de cierre de la compuerta Taintor. Para ello empleando andamios tipo PERI-ROSSET, y anclándola al paramento del vertedero, se preparó la citada plataforma a la que se accedía desde coronación a través de una escalera, desde un lateral del aliviadero, y circundando una pila en el caso del vano central.

El proceso de trabajo para instalar las ataguías comenzaría como se ha comentando practicando el hueco necesario entre las vigas. Con corte de disco de diamante se realizarían las líneas de demolición tanto de los vaineros laterales como de la viga de cierre, de este modo cuando se hormigonen los nuevos elementos, la junta de unión, entre hormigón viejo y nuevo queda perfectamente recta.

Con los cortes ejecutados se realizó la demolición del hormigón para practicar los huecos donde emplazar los elementos metálicos de viga de cierre y vaineros. Previo encofrado, se realice el hormigonado de los citados hierros fijos.

Llegado éste punto se pudo colocar las ataguías y comenzar el desmontaje de la compuerta Taintor, para lo que se procedió a su troceo empleando para ello oxicorte, y retirando los trozos obtenidos, con una grúa desde la coronación.

Figura 6. Ataguías colocadas durante los trabajos.

Una vez desmontada la compuerta Taintor se procedió a la retirada y nueva instalación de la viga de apoyo para el cierre de la compuerta, los sectores de cierre laterales, los ejes de la compuerta, y los apoyos de los cilindros hidráulicos.

Todavía quedaba por resolver un problema. ¿Cómo introducir e instalar las nuevas compuertas Taintor?

Podíamos fabricar las compuertas en taller en varias partes y soldarlas en obra, pero esto podía producir deformaciones debido a la soldadura. Por ello, y a pesar de ser compuertas bastante grandes y pesadas (15,00 × 4,75 m.) decidimos fabricarlas y transportarlas de una pieza. Esto tenía varios inconvenientes. Por un lado el acceso a la coronación de la presa era muy limitado, y la propia coronación, solo tenía 4 m. de anchura. Esto suponía una gran complicación para situar la grúa necesaria para manejar la compuerta con un peso de 16 Tn., ya que no había sitio para situar las patas de apoyo de la grúa. Y otra complicación era que la compuerta instalada en su lugar quedaba debajo del viaducto de los pasos sobre el aliviadero, es decir, la compuerta no se podía colocar directamente suspendida de la grúa, ya que debía quedar debajo del viaducto.

Para resolver todo esto, se decidió que la compuerta se transportaría en un camión góndola de pie, es decir en la posición en la que quedaría colocada. Realizando complicadas maniobras,

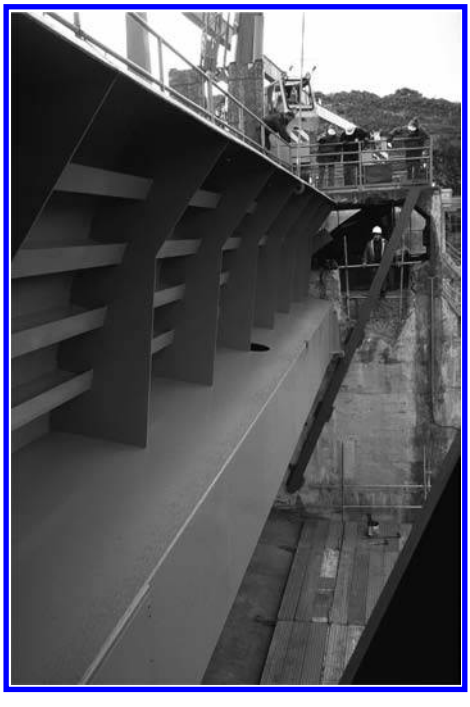

Figura 7. Montaje de la compuerta Taintor.

se podía introducir en la coronación de la presa el citado camión.

Para elevar y colocar la compuerta sería necesario el empleo de dos grúas emplazadas en coronación de tal manera que apoyasen las patas en las pilas tajamares y sobre las cámaras de maniobra de las compuertas, para lo que fue necesario instalar pilares metálicos en el interior de las cámaras, para soportar las cargas de las grúas. De este modo se situaba una grúa, luego se introducía la compuerta en el remolque, se retiraba la cabeza tractora del camión y se introducía la segunda grúa. De este modo se conseguía que las grúas quedasen lo más cerca posible de la compuerta.

Por último, para poder colocar la compuerta debajo del viaducto, se instalaron unas vigas carril fijadas a las pilas del aliviadero, de modo que guiaban a la compuerta, apoyada sobre ellas, en su descenso suspendida por las grúas hasta su emplazamiento definitivo debajo del viaducto.

Una vez en su sitio, se procedió a la instalación de todos los elementos restantes para el accionamiento, tales como cilindros hidráulicos, grupo oleohidráulicos, y las tuberías de aceite de accionamiento.

Una vez instalada la compuerta se la dotó de una pasarela de mantenimiento con barandillas de protección, que recorre toda la compuerta y a la que se accede a través de una escalera desde la cámara de maniobra. Como remate de la actuación se acondicionaron las cámaras mediante cambio de carpintería exterior, solado de terrazo y pintura de acabado.

Todo el proceso se repetiría en las otras dos compuertas.

4.2 *Sustitución de los desagües de fondo*

La actuación de reparación de los desagües de fondo era absolutamente necesaria, ya que por diferentes circunstancias se encontraban en la práctica, inoperantes.

Figura 8. Instalación de nuevas compuertas Taintor.

La presa de El Águeda tiene una gran acumulación de fangos en el vaso del embalse, que tras un estudio previo a la realización de los trabajos mediante investigación subacuática empleando buzos, demostró que las embocaduras de los desagües estaban cubiertas de fangos varios metros por encima de su parte superior. Por otro lado, los conductos de hormigón aguas abajo de las compuertas, presentaban un gran deterioro debido a las cavitaciones producidas por la salida del agua, por lo que seguir desaguando a través de ellos podía suponer un riesgo. Y por último las compuertas y mecanismos se encontraban muy deteriorados con un funcionamiento muy defectuoso, cuya maniobra de apertura y cierre podía resultar muy arriesgada.

Por todo ello, la única solución viable era la sustitución completa tanto de los elementos mecánicos como de los conductos de desagües. Como solución más idónea para los conductos se proyectó un blindaje metálico que revistiera todo el conducto, que una vez instalado se hormigonase al cuerpo de presa y se inyectase para garantizar su perfecto adhesión al hormigón de la presa.

Se estudió la posibilidad de hacer los trabajos vaciando el embalse, pero resultaba casi imposible garantizar el caudal ecológico del río durante la ejecución de los trabajos, ya que la fecha idónea para realizar la colocación del escudo, vaciando, sería el mes de octubre, y si una vez colocados los escudos no llovía, podía suponer un grave riesgo para la conservación de las especies piscícolas del río.

Por ello se adopto la solución más segura, pero también más delicada y dificultosa de colocar los escudos sin vaciar el embalse.

El proceso de ejecución de los trabajos, sería hacer primero un desagüe y una vez finalizado éste, comenzar con el otro. Los trabajos comenzarían con la colocación del escudo aguas arriba en la embocadura del conducto. Para ello habría que realizar en primer lugar, la retirada de los fangos y materiales acumulados delante de las embocaduras. Por los datos y planos antiguos que existían se pensaba que la parte baja de las embocaduras estaban justamente a la cota del terreno, por lo que para poder colocar el escudo sería necesario demoler picando al menos 40 cms por debajo de la embocadura.

El escudo se dimensionó de tal manera que una vez instalado se pudiera colocar el blindaje hasta la misma embocadura, incluyendo ésta, y por ello interiormente tenía una dimensión de 3,10 m. de altura, 2,40 m. de anchura y 1,00 m. de fondo. Disponía de una llave de paso neumática de simple efecto, que podía ser accionada desde coronación para su apertura en el momento de la retirada del escudo, para permitir el paso del agua e igualar presiones. También disponía de cuatro de cilindros hidráulicos para despegarlo de la pared en su retirada.

Comenzados los trabajos con los buzos se vio que realmente ejecutar aquel trabajo iba a ser tan costoso como se había supuesto.

Se trabajó con un equipo de buzos expertos en estas labores. Para extraer los fangos, se trabajo con una manguera de aspiración mediante un dispositivo ventury, a través del cual se impulsa aire comprimido y produce una fuerte absorción, (los buzos lo llaman coloquialmente "chupona"). Además de extraer materiales finos, iban apareciendo otros de mayor tamaño e incluso bloques más grandes de hormigón (se piensa que de cuando se construyo el aliviadero central de la presa, que fue de construcción posteriormente a la puesta en explotación de la presa), a los que hubo que realizar anclajes para extraerlos con grúa desde coronación. Una vez quedó limpio el fondo del desagüe, tal como se había supuesto, fue necesario realizar demolición en el fondo, para poder colocar el escudo. Estas demoliciones se realizaron con martillos neumáticos preparados para trabajar debajo del agua, martillos perforadores con barrenas, y quebrantador hidráulico (darda).

Finalmente se consiguió colocar el escudo.

Con el escudo colocado, comenzó la demolición en cámaras para extraer los elementos mecánicos viejos y preparar el hueco necesario para instalar los nuevos. En los conductos se procedió a sanearlos y prepararlos para introducir los nuevos blindajes de acero.

La demolición en las cámaras se realizó principalmente, con el empleo de un robot sobre orugas manejado por control remoto, y perfilando con martillos picadores neumáticos. Los conductos se demolieron en su totalidad con empleo de martillos picadores neumáticos y posterior lavado con bomba de agua de alta presión a 200 atm.

Finalizada la demolición se introdujo los blindajes y embocadura del tramo aguas arriba de las compuertas, emplazándolos en su lugar definitivo. Se introdujeron las compuertas Bureau nuevas de 1.200×800 mm. y se instalaron en el interior de la cámara y se continuó instalando el blindaje en el tramo de aguas abajo. Una vez estuvo todo instalado, se procedió al hormigonado de la totalidad del conjunto, conductos y cámara.

En el interior de los conductos se habían dejado unos manguitos preparados soldados al blindaje, para a través de ellos realizar la inyección de cemento necesaria para consolidar y unir el hormigón nuevo con el viejo. Esta operación se repitió varias veces, reperforando los taladros, hasta conseguir una perfecta unión. Una vez finalizada la inyección, se procedió a soldar y sellar unos tapones en los orificios preparados para la inyección quedando el conducto rematado para su puesta en funcionamiento. Se acondicionó el interior de la cámara de válvulas mediante revestimiento de láminas de PVC, i se instalaron los grupos hidráulicos, dejando la instalación lista para su puesta en servicio.

Figura 9. Cámara de los desagües de fondo demolida.

Figura 10. Desagües de fondo antes y después de las obras.

En el segundo desagüe se repitió el proceso expuesto en este desagüe.

5 CONCLUSIONES

Las obras, descritas sucintamente, han durado años, pero han llevado a El Águeda a una situación actual de homologación con una presa moderna.

La nueva galería, trabajo inabordable entonces sin el apoyo que prestó Guillermo, permitió descubrir y subsanar defectos desconocidos hasta entonces y permite ahora inspecciones y auscultación cercanas, resultando una vía más fácil para acometer actuaciones futuras.

Los nuevos órganos de desagüe y alivio abren un camino a la explotación de la presa con mucha menor presión para siguientes ingenieros encargados.

Retrocediendo mentalmente hacia atrás, y contemplando lo que ha costado definir cada proyecto, tramitarlo y ponerlo en práctica, hay que concluir que se necesita paciencia, constancia, ciertas dosis de insistencia y buenos apoyos alrededor, pero luego los resultados pueden ser magníficos. Por el camino se olvidan disgustos y se recogen enseñanzas, no siempre de fácil transmisión en las circunstancias actuales del dominio público hidráulico.

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Transversal crack repairs in Piaus dam

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ABSTRACT: The Piaus dam, located in the State of Piauí, Brazil, is a 45 m high earth dam constructed on a narrow V-form valley in a semi-arid region, where intermittent rivers flow only during the wet season (June–December). This type of climate makes it possible to build the dam without the need of river diversion works. The construction was initiated in June of 2008 and concluded in April of 2009, when the reservoir had a hydraulic height of 17 m (equivalent to a storage volume of 16.6 hm³). Three months after the completion of the dam, some transversal cracks were detected along the embankment: two were located near the left abutment and three near the right abutment. The larger crack on the left abutment was 3 to 5 cm wide and the one on the right abutment had a width of 1 to 3 cm. Exploration trenches indicated that the cracks extend 4.80 and 3.90 m below the maximum reservoir level (351.00). Aiming to seal these cracks before the rainy season of 2010, two trenches were excavated along the crest of the dam, completely surpassing the depth of the cracks. The trenches were filled with a mixture of soil and bentonite and the remaining portions of the cracks located up-stream from the trenches were gravity filled with a mixture of cement and bentonite, while the attempt to fill the portions located downstream from the trenches with sand were not successful. This paper will present the procedures of investigation and correction of the cracks on the Piaus dam, as well as the monitoring results of the dam's behavior with the increase of the water level in the reservoir during the rainy season of 2010.

1 GENERAL INSTRUCTIONS

The Piaus dam, situated in the State of Piauí, Brazil, with 45 m of height, and 104.5 hm³ of storage capacity, was constructed on a narrow V-form valley. The semi arid climate of the region, with rivers that do not flow in the dry season (June–December), allowed the construction of the compacted earth embankment without river diversion. The construction was initiated in June 2008 and concluded in April 2009, when the reservoir had a hydraulic height of 17 m (equivalent to a storage volume of 16.6 hm³). A month later, the reservoir reached 33 m, with a storage volume of 36.5 hm³.

In later June 2009, approximately three months after the completion of the dam, some transversal cracks were detected along the embankment: two were located near the left abutment and three near the right abutment. The cracks extended all the way through the width of the crest, being visible from the upstream slope riprap to the top of the downstream slope protection (coarse gravel). The larger crack on the left abutment was 3 to 5 cm wide and the one on the right abutment had a width of 1 to 3 cm.

Aiming to seal these cracks before the rainy season of the upcoming year of 2010, new 1.20 m wide trenches were excavated along the crest of the dam, completely surpassing the depth of the cracks. The trenches were filled with a mixture of soil and bentonite, compacted above optimum moisture content. After the backfilling of the trenches, as a complementary measure, the remaining portions of the cracks located upstream from the trenches were gravity filled with a mixture of cement and bentonite, and there was an unsuccessful attempt to fill the portions located downstream from the trenches with sand.

This paper will present detailed information about the procedures of investigation and correction of

the cracks on the Piaus dam, as well as the monitoring results of the dam's behavior with the increase of the water level in the reservoir during the present rainy season (January to June/2010).

2 PIAUS DAM

The Piaus dam is located in the Southeastern region of the State of the Piauí, about 300 km of the capital Teresina (Fig. 1), in the Marçal river, with coordinates 6°59'24"S, 40°50'12"W.

The State of the Piauí is inserted in the semi arid northeastern region characterized by intermittent rivers, shallow soil, potential evaporation exceeding precipitation and periods of recurrent droughts. These characteristics require the construction of dams for regularization of the rivers, water supply for populations and irrigated agriculture. The reservoirs vary in a large range of storage capacity, from a very dense array of small reservoirs for water supply of rural population to larger and medium dams that guarantee the water for urban areas and irrigated agriculture (DNOCS, 2008). The annual average precipitation of the region is presented in Table 1, where the rain concentration is shown from January to April.

The Piaus dam, with a crest extension of 333 m and maximum height of 45 m, was constructed in a narrow valley in a V-shape, with steep abutments with an average slope of 27%. The site of the dam is dominated by bare soil, with rare spots of vegetation, and sporadic outcrops of gneiss-granite rock at the bottom of the valley and the left abutment. This superficial soil is of weathered bedrock/colluvium origin, highly laterized, presenting gravel and irregular blocks of small sized rocks and quartzous nature.

The very bottom of the valley is flat and filled with sandy sediments of alluvium origin. Figure 2 presents a brief profile of the dominant lithological and geotechnical aspects of the valley along the dam axis.

The embankment is a zoned type earth fill, consisting of soil from borrow areas and from the spillway excavation; upstream slopes of 1:2 (V:H) from level 354.00 m to level 344.00 m, 1:2.5 (V:H) from level 344.00 m to level 334.00 m and 1:3 (V:H) from level 334,00 m to ground level. The downstream slopes are 1:2 (V:H) from level 354.00 m to level 344.00 m and 1:2.5 (V:H) from level 344.00 m to ground level. The geometry of the embankment is presented in Figure 3, which also presents the grain size curves of the construction materials.

The grain size, plasticity, permeability and shear strength properties of the embankment material

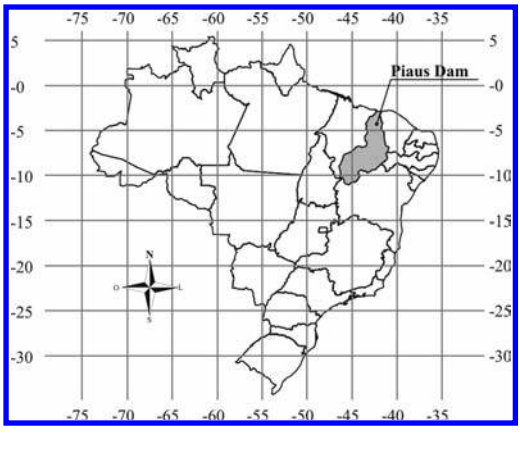

Figure 1. Location of the Piaus dam in Brazil.

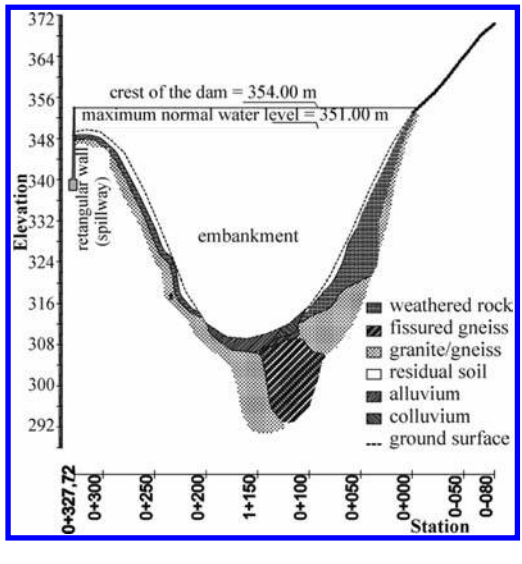

Figure 2. Geotechnical characteristics of the foundation.

Table 1. Averages of monthly and total precipitation (mm).

Jan	Fev	Mar	Apr	Mai	Jun	Jul	Aug	Set	Out	Nov	Dez	Total
81,7	110,0	166,8	116,0	36,5	10,6	4,3	2,7	3,9	10,8	25,3	44,2	612,6
13%	18%	27%	19%	6%	2%	1%	0%	1%	2%	4%	7%	
Total anual deviation: 263,7 mm												
Total anual CV: 0,43												

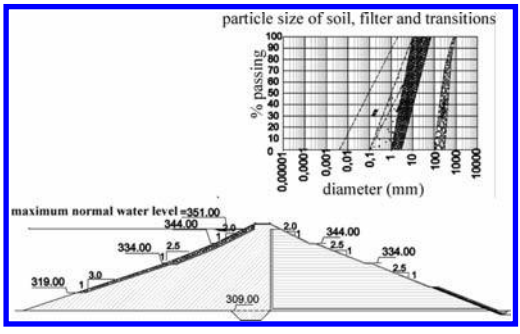

Figure 3. Section type of the embankment and grain size curves of construction material.

Table 2. Average values of the laboratory tests.

Material site	J-01	J-02	J-03	J-04
Laboratory tests				
#	Gradation (% passing)			
1”	100	99	98	100
3/8”	86	86	95	97
N°4	77	76	89	92
N°10	62	68	84	86
N°40	54	53	67	61
N°200	36	34	36	41
Atteberg limits (%)				
LL	32	31	34	36
LP	21	21	22	25
IP	11	10	12	11
Classification USC	SC	SC	SC	SC
Permeability (cm/s)	$1{,}0 \times 10^{-6}$		$1{,}1 \times 10^{-6}$	–

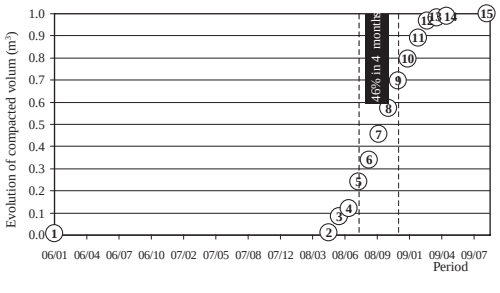

Figure 4. Embankment construction progress.

from the J-01, J-02, J-03 and J-04 borrow areas are presented in Table 2.

The embankment was built from May 2008 to September 2009 (15 months), as illustrated in Figure 4, however about half of the embankment volume was constructed during a 4 month period.

3 THE PROBLEM

The reservoir had not yet been submitted to a total filling when the cracks were first detected. In the end of April, 2009, in the final stage of construction when the crest level was reached (level 354.00 m), it was observed a transversal crack in the left abutment. The crack was very thin and less than 0.50 deep. At the time, the reservoir had approximately 17 m of water head (level 327.48 m). In May 2009, at the end of the rain season, it reached 36% of the total volume (33 m of water head and level 333.75) and more cracks were detected in June 2009.

These cracks developed throughout the crest, from the upstream slope to the downstream one, with the width of the cracks decreasing with depth. However, no longitudinal cracks or local depressions along the crest or over the slopes were identified. Also, the visual exam of the cast-in-place concrete curbs in the edges of the dam crest does not show any displacement in the upstream/downstream direction, neither abrupt vertical displacement in the cracks intersections.

Although these cracks have a limited depth, it is possible that they would be under water when the reservoir reaches its maximum operational level. In this situation it is very probable to appear seepages through the embankment. In general, the water would flow by the internal draining system and the loss would not be expressive enough to affect the dam operation. But, if the water flow surpasses the drainage capacity a very dangerous situation could take place, resulting even in piping and dam failure. Collected data from the National Research Council (1983) indicates that internal erosion was the strongest factor of collapse of earth dams with height superior to 15 m, in the period of 1900 to 1975, reaching 38% of accident causes in dams.

3.1 *Monitoring the cracks*

In late June 2009 two transversal cracks were detected in the left abutment, as well as three cracks in the right abutment (Fig. 5). The cracks developed across the crest, along its length, but it was not possible to follow their development in the embankment slopes since they were hidden by upstream rip-rap and by the gravel fill protection of the downstream slope. The greatest crack in the

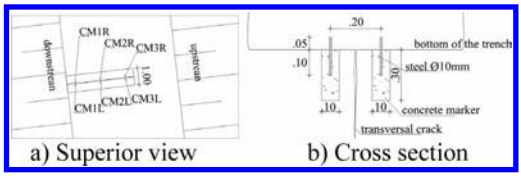

Figure 5. Location of the transversal cracks.

Table 3. Crack information.

Crack (n°)	Abutment	Classification	Station	Angle	Direction
1	Right	Secondary	0 + 46.90	36°53’	Left
2	Right	Principal	0 + 50.60	29°05’	Right
3	Right	Secondary	0 + 56.40	21°07’	Left
4	Left	Principal	0 + 242.75	45°24’	Right
5	Left	Secondary	0 + 250.80	23°42’	Right

left abutment has an width that varies from 3 to 5 cm and the one in the right abutment, from 1.0 to 3.0 cm.

Immediately after the identification of the cracks, a monitoring process of the dam was initiated by the control of surface monuments and the measurement of the cracks widths. Crack information is presented in Table 3, and their location in Figure 4.

At the same time, to investigate the development of the main cracks in depth, inspection trenches were manually opened in the right and left abutments (cracks n°2 and n°4). The trench in the left abutment (crack n°4) reached 8.5 m in depth, and the crack was visible up to 7.8 m, presenting a vertical deviation of 7° in the abutment-river direction in the first 4.8 m, becoming vertical after that. In the trench of the right abutment (crack n°2) it was possible to verify that the crack disappeared after 6.9 m of depth and shows the same behavior of vertical deviation. In this case, with a vertical deviation of 2° in the first 4.0 m. So, the cracks reach the levels 346.20 and 347.10 m, respectively, meaning they were still open at 4.80 m and 3.90 m below the maximum level of the reservoir (351.00 m).

To monitor the cracks development, in order to guide the dam repairs, gauges consisting of pairs of concrete embedded steal rods were installed on both sides of the cracks to measure surface displacements. The distances between the steal rods were measured with a millimeter scale always at the same time of the day, 7 am, to avoid the high temperatures that occur in that region during the day. The rods on the right abutment (crack n°2) were damaged during the execution of the trench. Since then, this crack development was followed only by the visual control of the cast-in-place concrete curb in the downstream edge of the dam crest. Figure 5 shows the instrument details.

Figure 6 illustrates the results of the surface displacement measurements; where it can be seen that the opening of the main crack on the left abutment stabilized since 14/08/2009. Therefore, it was assumed that the cracking process had stabilized and the repair works could be done.

Since the anomalies had been detected, inspections on the dam were carried out in order to evaluate its

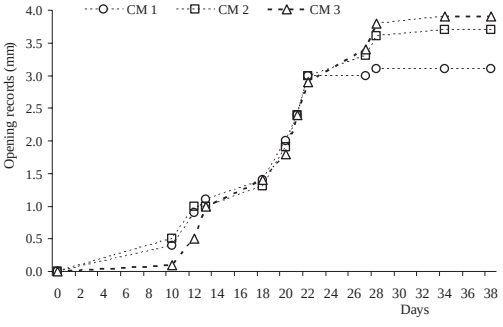

Figure 6. Monitoring records for four gauges.

performance and to search for other defects. From these inspections it can be concluded that:

- No more new cracks appeared in the crest of the dam;
- There are no evidences that could associate the cracks on the left abutment to the outlet works proximity;
- The embankment, as can be seen in the inspection trenches, is homogeneous and well compacted, without discontinuity between compaction layers;
- Wet zones that could be evidence of seepage outflows through the embankment had not been observed on the downstream slope

The inline equations (equations within a sentence) in the text will automatically be converted to the AMS notation standard.

4 BRIEF DIAGNOSIS

After Sherard et al. (1979) and Casagrande (1950), cracks are anomalies generated from different types of settlement, they could be transversal or parallel to the dam axis. They can be associated to the geometry of the valley, the relative compressibility of the foundation, and the embankment properties itself. And the CIBD/ICOLD (2000) defines cracks as anomalies which can reach up to 15 cm of width, being more common from 3 to 5 cm; the largest number of occurrences is in dams

higher than 30 m. In large dams they occur close to the embankment top. The most critical situation occurs when the foundation at the bottom of the valley is much more compressible than the foundation at the abutments.

In the Piaus dam there is a conjunction of the above factors presented by Sherard et al. (1979) and Casagrande (1950), resulting in a situation that fits the definition of the CIBD/ICOLD (2000), since:

- The valley is very narrow, in a form of a "V", with very steep abutments and the dam is higher than 30 m;
- The foundation material in the center of the valley is more compressible than that in the abutments;
- The embankment soil, that presents IP < 15, was compacted in the dry side of the compaction curve;
- The detected cracks are 3 to 5 cm wide;
- The high construction speed of the embankment resulted in the compaction of great volumes of earth fill in a short period of four months.

It is not uncommon that dams, even if constructed with rigorous compaction control, present cracks, but most of those occurrences are not reported, nor investigated. On the other hand, a crack can be "hidden" for a long time without being detected, different from an embankment slide that is easily perceived. More yet, it is necessary to consider that it is not easy to identify the causes of the cracks.

In accordance with Sherard et al. (1979), there is a possibility of seepage formation from cracks. Most of the cracks occur in the first filling of the reservoir and some of them are due to pipings located next to outlet works or in the embankment-foundation contact. The case of the Apishapa dam in the State of Colorado, USA, (Romain, 2007) is an example of a dam constructed over foundation that is much more compressible in the center of the valley than in the abutments. In this case, initially, it was observed a significant settlement of the embankment in the first filling, followed by a transversal crack and seepage in the downstream slope of the dam and in the embankment-abutments contact. The settlement process continued resulting in the dam collapse, two years after the first filling

5 THE REPAIR

Four alternatives have been examined to repair the cracks of the Piaus dam:

- Removal of the material at the upstream zone of the embankment on the area of the crack and replacement with clay soil compacted at water content above the optimum;
- Open trenches in the area of the crack and fill them with plastic concrete slurry (mixture of cement, bentonite and water);
- Injection of the cracks with plastic bentonite-cement mortar;
- Open trenches in the area of the crack and fill them with clay soil compacted at water content above the optimum.

The first alternative demands that the excavation reaches 9.0 m depth under the crest, what would cause a strong impact in public opinion. The opening of a trench and its filling with plastic concrete slurry (second alternative) could damage the vertical drain, more yet, would require expertise for the attainment of a concrete that would be at the same time impermeable and have strength-deformation behavior similar to the embankment. The third alternative also presented the disadvantage of a possible contamination of the sand of the vertical filter. Therefore, remains the fourth alternative that can be summarized as the excavation of a trench that is backfilled with plastic soil, avoiding the impact of large excavations in the crest and, at the same time, without the need of specialized labor skills or equipments that are not available in the region.

The only restriction would be the maximum digging depth of the hydraulic excavators available in the region which are limited to 5 m, not capable to reach the 9 m depth necessary to surpass the cracks. This problem was solved with the excavation of part of the embankment 4 m bellow the crest. From these factors resulted a four phase methodology (see Figure 7), as follows:

1st stage
To comply with the maximum digging depth of the hydraulic excavator it was excavated an access trench along the dam axis with 4.5 m width and 4 m depth (bottom level 350.00 m).

2nd stage
Opening a deep trench with 1.2 m width and 5.0 m depth, starting on the bottom of the access trench. Manual backfilling of the deep trench with a mixture of borrow area soil and bentonite (2.5%), moisture above the optimum (1% plus). The mixture of the soil and bentonite was compacted in 15 cm layers. The geotechnical characteristics of the soil and the mixture are presented on Table 4.

3rd stage
The remaining portions of the cracks, inside of the access trench and located upstream from the deep trench, were gravity filled with a mixture of cement and bentonite. The injection holes had been opened at the bottom of the access trench, with 6"diameter trade, spaced with 0.5 m distance along the cracks. These holes followed the inclination of the crack

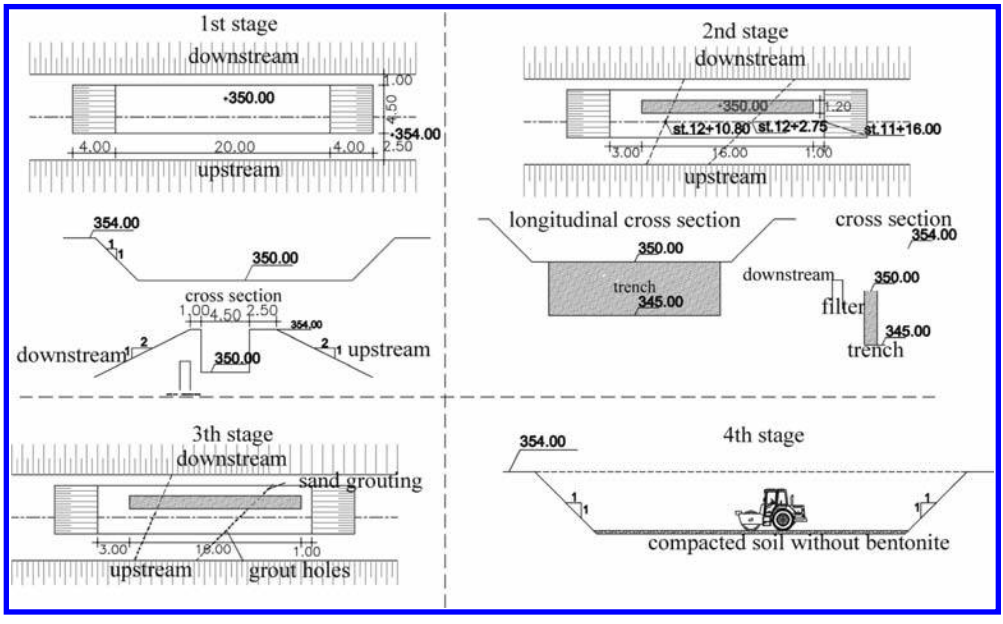

Figure 7. Stages of the cracks repair.

Table 4. Geotechnical characteristics.

Characteristic	Soil	Soil- bentonite (2.5%)
LL (%)	31	39
PI (%)	9	13
DD max (N/m^3)	1938	1820
WC optimum (%)	11.2	11.8
# 200 (%)	44	44
Permeability (cm/s)	2.5E-07	8E-08

in relation to the vertical. The holes were injected from the deep trench to upstream.

The mixture proportion of the bentonite-cement grout was defined experimentally in laboratory in order to attain a steady suspension, with low exudation, high viscosity, low resistance and rigidity when compared with cement grout. The proposal was to obtain properties close to that of the embankment. Initially mixtures based on similar experiences (ICOLD, 2000) were tested, as those of the restoration of Balderhead dam, Coulder dam and Widerford dam, which bentonite percent in the mixture varied from 4 to 11% and cement from 11 to 23% in weight. However, in the case of the Piaus dam, for the grout to be applied without pressure, only by gravity, the best results had been obtained with 75% of water, 18% of cement portland and 7% of bentonite.

The injection consisted of filling the holes with grout in regular intervals of 10 minutes and taking note of the added volume. This process was repeated until the absorption was less than 1.0 cm, after 30 minutes (6x10–3 l/min). Following this process, the total consumption of grout for each crack resulted: crack n.°1 = 28 liters, crack n.°2 = 586 liters, crack n.°3 = 130 liters; crack n.°4 = 535 liters, crack n.°5 = 112 liters. Grout was not observed seeping out in the upstream rip-rap.

There was an attempt to fill the part of the cracks, situated downstream of the deep trench, with fine sand in water suspension. However, the intended filling did not succeeded. The sand in water suspension did not seep through the cracks, accumulating itself into the holes opened to be injected.

4th. stage

Finally, the access trenches were backfilled with borrow area soil, the same as used in the dam construction the compaction curve and soil with plasticity index less than 15%.

6 CONCLUSION

Transversal cracks presented in the Piaus dam, near the abutments, had been detected after the first partial filling of the reservoir. An evaluation of the situation confirmed the existence of a conjunction of factors pointed in literature as associated to the appearance of this type of crack. These factors are: very narrow valley, foundation in the valley bottom more compressible than that in the abutments, high construction speed, soil compacted in the dry side of

The repair process consisted of opening an access trench, followed by digging a deep trench that was later back filled with a mixture of soil and bentonite (2,5%), complemented with the injection of cement and bentonite grout in the cracks upstream of the deep trench. The execution was simple, using the equipment that was available in the region and without requiring special labor skills, at the same time preserving the integrity of the internal draining system of the embankment. The adopted grout, with 75% of water, 18% of cement portland and 7% of bentonite (in weight), resulted in a steady suspension with low exudation, good workability and, when applied in atmospheric pressure, presented a fluidity enough to penetrate deep into the cracks.

Since the repair of the cracks, the reservoir level has raised only 40 cm due to a poor rainy season in 2010 with precipitations below the historical average. Even so, an inspection of the dam was carried out last May to verify the behavior of the embankment. At that time, no evidences of new cracks in the embankment were found. The topographical control of the surface monuments on the dam crest, in the last six months (late November-early May), showed no differential settlement in the area where the cracks had been detected and re-paired, even with a maximum settlement of 7.0 cm in the center of the valley.

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Rehabilitation of Serra dam (Switzerland) affected by ASR

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ABSTRACT: The concrete of Serra dam, completed in the year 1952, is affected by Alkali-Silica Reaction (ASR) leading to a rising of the dam crest of about 25 mm and to an irreversible upstream displacement at dam crest of more than 60 mm at the end of 2009. Due to the various cracks in the concrete, the safety of the dam can not be anymore guaranteed. The paper illustrates the behavior and the damages of the existing dam due to concrete expansion. After discussion of the three solutions proposed for the final dam rehabilitation, the relevant features of the selected alternative, consisting in the construction of a new dam, a few meters downstream of the existing one, are presented. The final design was submitted to the competent swiss federal authorities in 2008, while the construction started in September 2009 and is planned to be completed at the end of 2010, including the demolition of the old dam.

1 INTRODUCTION

The Serra dam is located in the Zwischbergen Valley, on the southern side of the Simplon Pass near the village of Gondo (Canton of Wallis, Switzerland). The existing arch dam was constructed during the period 1951–52 in order to guarantee a daily storage to supply the hydroelectric power plant of Gondo. The plant, owned by the Energie Electrique du Simplon SA (EES), is equipped with three Pelton turbines, which, with a maximum operating flow of 11.5 m^3/s and a total power capacity of 45.4 MW, guarantee an average annual production of 177.3 MWh.

The concrete of the dam exhibited signs of expansion due to an alkali-silica reaction which, over the years, led to a irreversible displacement toward the upstream direction and a heaving of the dam crest. These displacements caused scattered cracks in the dam, especially along the downstream perimeter, which have progressively led to a deterioration of the operating and security conditions of the dam. Following this situation, a rehabilitation of the existing dam had to be carried out.

After studying various solutions, the most appropriate one was found to involve the construction of a new dam a few meter downstream of the existing one. The construction work is currently ongoing and should be completed by the end of 2010. After the completion of these works, the existing dam will be demolished.

2 CHARACTERISTICS OF THE EXISITING DAM

The existing Serra dam was built in a vertical cylindrical arch structure with a maximum height of about 20 m from the lowest foundation elevation.

This type of structure can be recognised in other small dams constructed in the 1950s. Along a horizontal plane, the dam is constructed by a series of circular arches, whose radii vary from 28 m in the central section to 170 m near the banks. The upper arches have a constant thickness of 1.0 m, while the thickness at the foot increases progressively

Figure 1. Downstream view of the Serra dam.

reaching a maximum value of 3.5 m. The dam was constructed in 6 independent blocks approximately 11 m wide, separated by 1 m wide contraction joints. These joints were concreted the year after the completion of the main blocks, when concrete was completely cooled down, without carrying out any additional grouting along the joints.

The rather unfavourable design of the dam required a steel reinforcement along the whole downstream face as well as at the upstream foot of the foundation in order to resist tensile stresses (Fig. 2).

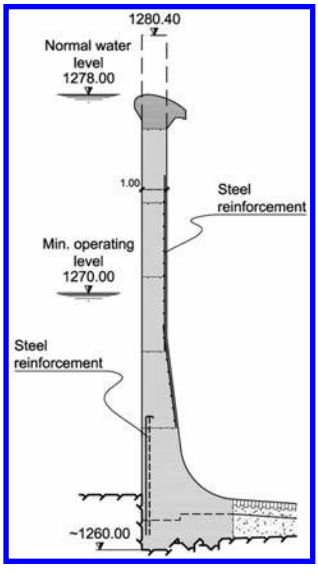

Figure 2. Typical cross section of the existing dam.

The crest of the dam is located at an elevation of 1′280.40 m a.s.l., while in the central section there is a 32.60 m wide spillway with the sill at elevation 1′278.0 m a.s.l. The spillway was designed for releasing 175 m^3/s at the maximum reservoir elevation (1′279.84 m a.s.l.). When the reservoir level rises up to the crest, the spillway reaches a capacity of approximately 260 m^3/s.

The dam is equipped with a bottom outlet at the right bank, whose axis is located at 1′264.25 m a.s.l. The bottom outlet, with the entrance located in front of the headrace tunnel intake, is composed of a circular pipe 1.10 m in diameter controlled by a submerged sluice gate, which is installed on the upstream face of the dam.

For the construction a diversion tunnel, with a capacity of 40 m^3/s and by-passing the whole impound, was excavated on the right bank in order to protect the site against flooding. This tunnel is still used to divert the river and limit the sedimentation in the impound during floods.

A total concrete volume of 2′300 m^3 was used for the construction of the dam. The dam has a slenderness coefficient C (defined by Lombardi as $C = S^2/(V \cdot H)$, where S is the developed surface, V the concrete volume and H the maximum dam height) equal to 40, which is a rather high value compared to other arch dams under similar condition.

With the exception of the spillway sill, the concrete of the dam had a dosage in Portland cement of 275 kg/m^3, to which 55 kg/m^3 of fly ash as well as an aerator to improve its frost resistance characteristics were added.

The foundation rock is composed of calcareous schist with interbedding of dolomite belonging to

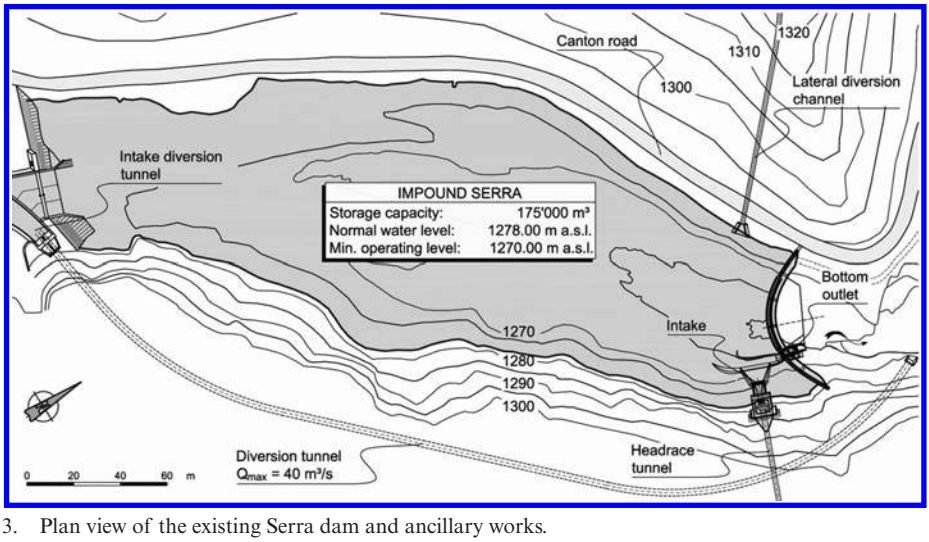

Figure 3. Plan view of the existing Serra dam and ancillary works.

the Monte Leone formation. The rock was treated using cement grouting down to a depth of 15 m. The average absorption of 123 kg/m of borehole revealed a certain degree of fracturing in the rock mass.

3 STATE OF CONSERVATION OF THE EXISTING DAM

3.1 *Behaviour of the dam*

The displacements of the dam have been checked by means of geodetic measurements since 1952, when it was put into operation. The measuring network is composed of 16 target points located on the downstream face of the dam.

Permanent displacements of the dam began to be apparent beginning in the 1970s. In Figure 4, the total vertical displacements measured until 2009 are presented. It might be noticed how the vertical expansion is generally proportional to the height of the blocks with respect to the foundation.

In the central part of the dam, the total movements exceed 20 mm. Taking into account the height of a measuring point of 15 m, a displacement of approximately 1.5 mm per meter of concrete is obtained, which over a period of 40 years corresponds to an average annual expansion of about 35 μm/m. Figure 5 shows the horizontal displacements in radial direction as a function of time for some characteristic points of the dam. The maximum displacement of 65 mm was measured at crest level, while the movements corresponding to the arch at the height of the spillway attained 50 mm. Such displacements correspond to an average annual expansion of 32 μm/m.

In thermal equivalent terms, the displacement of the dam measured over a period of 40 years may be compared to expansions produced by heating the concrete by approximately 150°C. In addition, it

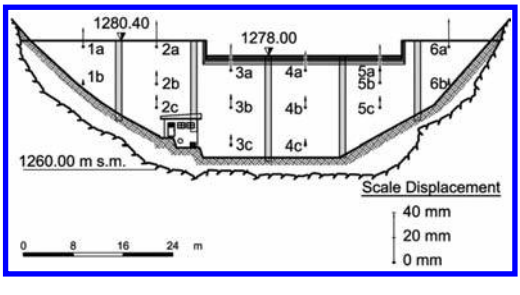

Figure 4. Total vertical displacements of the downstream dam face measured from 1952 to 2009.

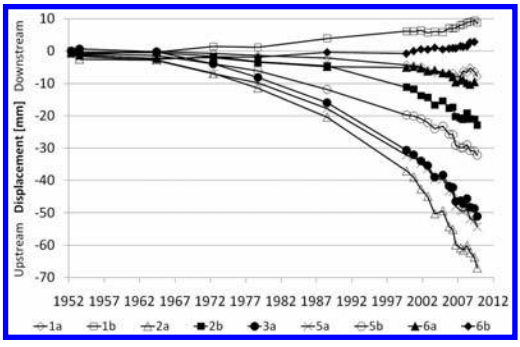

Figure 5. Horizontal displacements of some characteristic points on the downstream dam face from 1952 to 2009.

Figure 6. Peripheral crack at the downstream toe of the dam (detail over the gate chamber roof).

may be noted that, over the last years, a progressive acceleration of the movements in radial direction occurred (Fig. 5).

Large irreversible movements due to concrete expansion led to a state of cracking spread out over the dam. A detailed survey permitted to quantify the total length of the cracks present on the downstream face to about 440 m and those on the upstream face to about 220 m. Several thin cracks were observed along the horizontal lift joints. A peripheral crack at the downstream toe of the dam turned out to be of particular importance (Fig. 6).

It is interesting to note that this crack is located at the same level as the upstream limit of the foundation rock. Figure 7 shows the probable mechanism leading to the formation of the peripheral crack. The extension inside the dam of the peripheral crack could not be exactly determined.

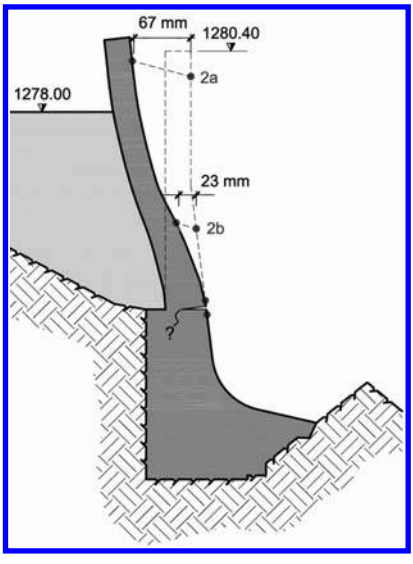

Figure 7. Mechanism at the origin of the peripheral crack.

3.2 *ASR reaction of the dam concrete*

Some studies carried out in 1991 an 2002 already confirmed that expansion and cracking of the existing dam is related to an Alkali-Silica-Reaction (ASR) of concrete. In view of the study of alternatives for the final rehabilitation of the dam, some additional studies were carried out at the Swiss Federal Institute of Technology of Lausanne (EPFL).

In order to evaluate the potential of a rehabilitation of the existing dams or a reconstruction of a new dam on the existing foundations, further knowledge on the future behaviour of the concrete was required. The main purposes of those new tests might be summarized as follows:

- provide information on the state of advancement of the ASR and on the residual reactivity potential of the aggregates respectively;
- characterization of the expansion process kinematics as function of the temperature and of the alkalinity. With respect to the temperature, the purpose was to determine the activation energy, which is the basic parameter in order to evaluate the concrete expansion at the effective temperature of the dam;
- predict the expected variation of the mechanic properties of the dam concrete with the progress of the ASR.

Two types of aggregates were used for the investigations. The virgin aggregates coming from the neighborhood of the reservoir (supposed to be the same used for the dam construction), and the aggregates extracted from concrete cores taken from the dam.

These aggregates were used to perform microbar tests at 150°C as well as swelling test on mortar prism at three temperatures (38°C, 50°C and 60°C) with various alkali contents. Mechanical tests were also carried out on these samples (flexion and compression test, Young's modulus determination) .

The aggregates reacted fraction in the mortar samples were also determined at different expansion rates by means of SEM back scattered electron images.

Against expectation, it was found that virgin aggregates already contain between 3.0 and 4.5% of ASR reacted fraction, that is approximately the same reacted fraction observed in the core extracted fraction. The microscope acquisition images confirmed the presence of cracks in the virgin aggregates at the beginning of the tests. Probably, the virgin aggregates were exposed to a wet and alkaline environment before testing. The alkaline ion sources might, for instance, come from the presence of potassic feldspars in the stones.

The comparison between core extracted aggregates (having already reacted during several years in the dam concrete) and virgin aggregates collected in the dam region shows very similar expansion behaviour and did not lead to decisive conclusions about the residual expansion potential of the dam concrete.

However, the high expansion observed at the core-extracted aggregate mortars allows to assume an important residual reactivity potential of dam concrete.

The mechanical test results show that the Young's modulus decreases with increasing the reacted fraction of aggregates, while the flexural residual strain decreases under the same conditions and the compressive residual stress remains stable at all ASR states.

3.3 *Structural safety of the dam*

The structural safety of the existing dam was verified using various approaches. First, stresses and displacements in the dam in its original state were determined. This verification revealed the presence of very low stresses.

Then, a simulation of the state of the dam with concrete expansion was carried out. The calculation was performed for three different hypotheses. The first corresponded to a perfectly elastic model with no cracks. The second considered the presence of a crack along the perimeter of the dam. Finally, the presence of a peripheral crack and a certain horizontal cracking was simulated, which in fact reduces the contribution of the cantilevers. In all three cases, very high tensile stresses were obtained (from –7 MPa to –12 MPa), which are not compatible with the concrete characteristics.

Thus, it was also deemed necessary to verify the case of complete horizontal cracking, by assuming

the dam as a sequence of independent arches, without taking into consideration any contribution from the cantilevers. This calculation showed that even in this extreme situation of cracking, the existing dam is able to support the entire hydrostatic load with sufficient safety.

Finally, the stability conditions of the foundations were verified, assuming a possible sliding of the dam along the peripheral crack.

In summary, the results of the static analyses of the existing dam were the following:

- the maximum stresses in the initial state of the dam without expansion are very low, with a maximum compression value reaching 1.9 MPa and tensile forces in the order of –1.2 MPa;
- taking into account the expansion and a non-cracked elastic modulus, the maximum stresses reach 34 MPa in compression and –13 MPa in tension;
- with the simulation of a cracked state of the dam, relatively low tensile and compressive stresses are obtained. Therefore, dam collapse due to exceeding the compressive strength of the concrete can be excluded;
- even assuming a complete horizontal cracking of the dam (model with independent arches), the structural security is not in doubt;
- the most plausible mechanism for a possible collapse of the dam is due to sliding along the peripheral crack. Nevertheless the stability conditions become critical only in the case of a degree of cracking exceeding 90% of the surface of the dam.

3.4 *Hydraulic safety*

During an exceptional flood in October 2000, an overflow at the crest was observed. The peak flow rate during this flooding event was evaluated to be 320–340 m^3/s, thus much greater than the design flood value of the existing spillway.

On the basis of an update of the hydrological data, the design flood value for the Serra dam (1000 years return period) was estimated at 540 m^3/s. According to Swiss regulations, the dam must be able to evacuate PMF safely without any danger to its stability.

The spillway of the existing dam is therefore not able to assure the required hydraulic safety according to the new hydrological assumptions.

4 PROPOSALS FOR DAM REHABILITATION

4.1 *Rehabilitation options studied*

Three options were studied for the final rehabilitation of the dam (Fig. 8):

1. Rehabilitation of the existing dam:
   This solution provides, in a first step, vertical cuts made by diamond wire in the centre of each segment over approximately half of the dam height. In this way the internal stresses that were created following the expansion of the concrete are reduced and the conditions similar to those in the initial state are re-established. Subsequently, these incisions are grouted in

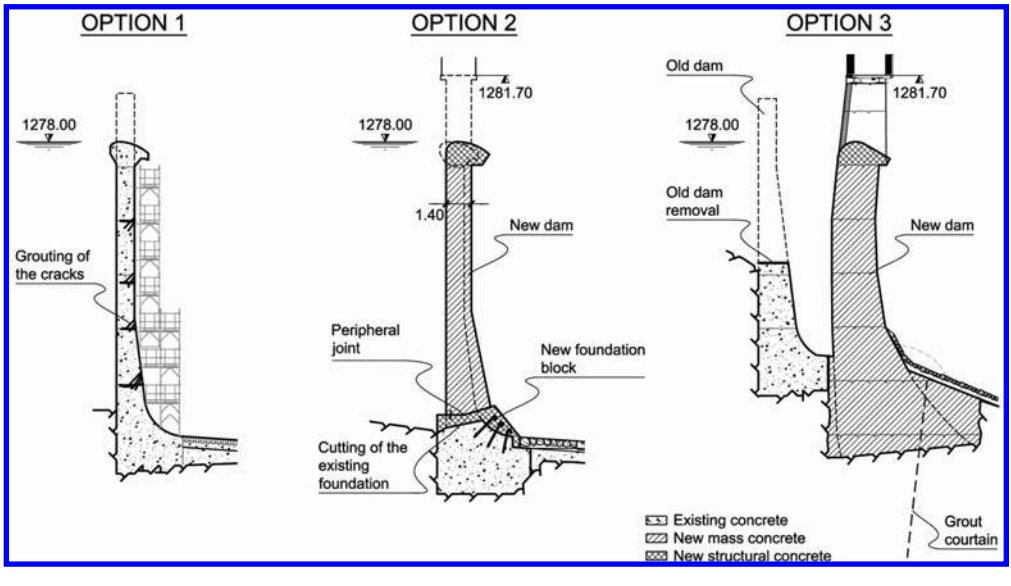

Figure 8. Options evaluated for the final rehabilitation of Serra dam.

order to restore the arch effect. Finally, grouting of all cracks are carried out using epoxy resin to guarantee a continuous structure again.

2. Reconstruction of a new dam on the existing foundations:
   The solution provides the demolition of the existing dam down to a level lower than that of the peripheral crack and to reconstruct the dam on the existing foundations. The new dam is supported by a foundation block including a portion of the existing foundations. Between the foundation block and the dam, a peripheral joint (pulvin) is designed, whose primary aim is to avoid tensile stresses at the base of the dam and in particular at the contact between the new and old concrete in the foundation block. From a geometrical point of view, the new dam must be identical to the old one; only the thickness is increased from 1.0 m to 1.4 m in order to improve stability conditions.
3. New dam at the toe of the existing one:
   The third and last solution suggests the construction of a new dam located downstream of the existing one. The topographical conditions, especially the presence of a cliff some tens of meters downstream of the existing dam, dictate the position of the new dam to be immediately adjacent to the old one.

4.2 *Choice of the optimal option*

The first option, which is economically the most favourable, was dismissed from the beginning as it is conditioned by too many unknowns concerning the medium and long-term behaviour of the dam. In fact, it was not possible to determine with any certainty the residual expansion potential of the concrete. A rehabilitation of the existing dam could therefore not be considered a final solution. The continued advancing of the expansion would make periodical vertical cuts in the dam necessary to decompress the structure before new cracking. The frequency of these works cannot be evaluated in advance.

From a safety and serviceability point of view, both options 2 and 3 give sufficient guarantees. Option 3, however, implies higher costs, especially due to the excavations of the new foundations, the increased concrete volume and because of the need for additional grouting works in the foundation (grout curtain). The reasons for recommending option 3 are primarily associated with the possibility of keeping the existing dam in service during the construction works. This permits the construction of the new dam during the summer, which in the alpine region corresponds to the period with the highest water discharges.

If option 2 would be adopted, the construction works would have to be carried out during the winter months in order to limit production losses. This would increase the difficulties due to low temperatures and winter weather (avalanche risk along the access road).

Furthermore, the construction of a new foundation allows some project freedom, including the choice of a more favourable dam geometry compared to that of the existing one.

5 THE NEW SERRA DAM

5.1 *General characteristics of the project*

The new dam is to be constructed adjacent to the existing dam, as shown in Figure 9. Such an arrangement is mainly defined by topographical conditions. In fact, a deep cliff several tens of meters downstream from the existing dam precludes to move the new dam any further downstream. In order to improve the angle of intersection between the arches and the right bank, it was decided to rotate the axis of the dam slightly clockwise with respect to the existing dam. Consequently, the new dam intersects the existing structure at the right bank.

From a structural point of view, the new dam can be assimilated by an arch dam with double curvature. Considering the limited dimensions of the structure, it was decided to opt for a simple geometry in which the curvature is defined vertically using a base line with changing direction corresponding to the concreting joints. Horizontally, the arches of the dam are of elliptical shape. The new dam is designed with 6 independent blocks, whose widths vary between 12.0 m and 14.4 m. These blocks are separated by contraction joints, which will be grouted with cement slurry after concrete cooling.

The crest of the new dam is designed at 1282.70 m a.s.l., i.e. 1.30 m higher than that of the existing structure. The total height of the new dam is 22 m, while the total concrete volume of the dam is 3′500 m^3. The new dam has a rather low

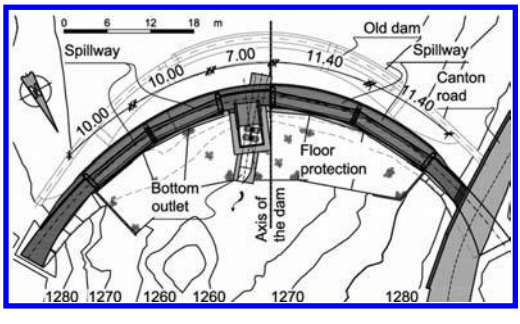

Figure 9. Plan view of the new dam.

slenderness coefficient C equal to 17. However, it has to be noticed, that the new dam is designed to allows the structure to be eventually further raised by additional 6 m in order to increase the storage volume of the reservoir.

For the concrete of the new dam no-reactive aggregates were selected in a quarry about 50 km far away from the site. In order to further reduce the risk of ASR the concrete mix design foresee the use of a blast furnace cement CEM III/B 42.5 (dosage of 220 kg/m^3).

The static calculation of the dam confirms a low stress state, with stresses attaining 2.0 MPa in compression and 0.5 MPa in tension considering only static loads. Taking into account the thermal effects, maximal tension stresses of −1.4 MPa are calculated.

5.2 *Bottom outlet*

Contrary to the existing dam, where the bottom outlet was positioned along the right bank close to the water intake of the headrace, the new bottom outlet is arranged at the centre of the valley. In this way, the efficiency of the flushing operations, which are normally carried out annually, should be improved.

The bottom outlet is designed as a square conduit of 1.60 m × 1.60 m, with a longitudinal slope of 10%. The upstream stretch, up to the two sluice gates (for revision and service), is entirely steel lined while the downstream stretch is made of an open channel. At the end of the channel, a deflector allows high energy dissipation.

The valve chamber for the sluice gates is located at the downstream toe of the dam. The access to the chamber is a vertical shaft fixed to the downstream face of the dam (Fig. 10). This rather inconvenient solution has the advantage of guaranteeing access to the room even during flood events.

To reduce the structure dimensions at the base of the dam, a lifting system was not to included inside the chamber. For the mounting and dismounting of the hydromechanical equipment, the installation of a jib crane on the crest and an opening in the roof of the valve chamber with a removable cover is foreseen.

5.3 *New spillway*

The new spillway is designed for a much greater flood flow rate than that of the old dam. Such an increase in capacity was accomplished by increasing the total length of the sills and the crest elevation of 1.30 m with respect to the existing dam.

The spillway is essentially composed of 2 fixed sills 9.6 m wide on the right bank and 2 fixed sills 12.0 wide on the left bank (Fig. 9). The total length of the spillway is therefore 43.20 m. The sills are

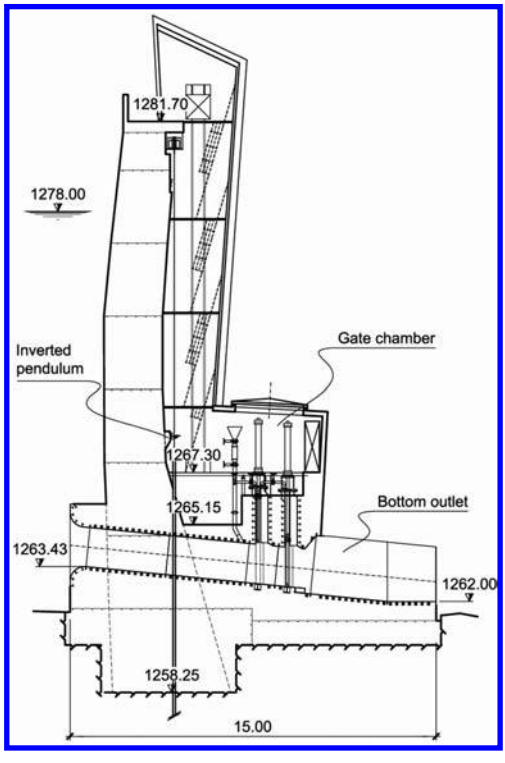

Figure 10. Typical cross section of the new bottom outlet.

surmounted by metallic walkways which link both banks and the valve chamber.

5.4 *Monitoring instrumentation*

The monitoring instrumentation is essentially concentrated in the central section of the new dam, where the horizontal movements are measured using an inverted pendulum installed in the access shaft for the valve chamber. The pendulum has two measuring stations; the first at crest level and the second at about mid-height (Fig. 10). Electric thermometers are installed at three different levels in the same section of the dam to permit the interpretation of the displacements.

A piezometer to measure uplift pressure in the dam foundations is also foreseen.

The monitoring device is completed by a geodetic measuring network and a levelling at the dam crest.

6 IMPLEMENTATION OF THE CONSTRUCTION WORKS

In this chapter some aspects of the dam construction phase are presented. At the moment of

writing of the present paper, the works are still in a preliminary phase, consequently the presentation is incomplete.

6.1 *Excavation of the foundation*

The excavation of the foundation at the toe of the existing dam, which is still in operation during works, represent one of the most challenging aspects of the work. Severe vibration thresholds (vibrations are continually monitored using geophones installed on the crest of the existing dam) were defined to avoid damage to the still operating dam.

In order to limit the vibrations due to the excavation phase, first of all, vertical cuttings are made using diamond wire at the upstream and downstream edges of the foundation trenches. Once these cuttings, which are generally carried out in 10 m long stages, are completed, the primary excavation is undertaken. Initially, the excavation was to be carried out using explosives. Even if only a minimum quantity of explosives was used, it was not possible to keep the vibrations within the desired limits.

Therefore, it was decided to carry out the excavations using a pneumatic pick mounted on a digger, which permitted sufficient limitation of the vibrations but also slowed down the progress of the excavation work (Figs. 11 and 12).

6.2 *Stability of the existing dam during the construction*

A check of the stability of the existing dam revealed insufficient safety coefficients in the zone of the right bank, where carrying out the excavations requires a partial demolition of the downstream toe of the existing dam (Fig. 13).

For this reason, it was necessary to lower the reservoir level by 4 m during the construction work. However, such a measure is not sufficient, since an increase in water level due to a flood event cannot be excluded.

In order to guarantee sufficient safety margins, it was therefore necessary to place 35 anchors bars Ø 32 mm of length varying from 5 to 8 m in the foundation of the existing dam.

Furthermore, in order to protect the excavation of the foundation against flooding, these works were scheduled in spring, before snow melting.

6.3 *Protection of the work site in case of flooding*

The diversion tunnel of the reservoir and the operating flow in the powerhouse permits a discharge flow of about 50 m^3/s without causing any increase in the pond level. However, considering a flood rate

Figure 11. Excavation of the dam foundation.

Figure 12. Excavation of the foundation by the central block.

Figure 13. Stabilization of the downstream toe of the existing dam on the right bank.

with a 10 year return period (estimated at 150 m^3/s), an overflow of the existing dam cannot be avoided. Lowering the retention level about 4 m during the construction period guarantees a storage volume of about 80′000 m^3 in the reservoir, which allows a certain reaction time in case of flooding.

The owner decided to implement an hydrological prevention model in order to prevent any flood risk of the work site with sufficient warning. This model is based on meteorological forecasts and permit an evaluation of the flow which arrives in the basin with 24, 72 and 240 hours notice respectively.

6.4 *Concreting of the dam*

The construction of the 6 dam blocks are generally carried out by 3 m high concrete lifts. The first block on the left bank was poured in autumn 2009, whereas the five blocks remaining are scheduled to be completed during 2010. A total of 30 concrete lifts are necessary to complete the remaining blocks. Taking into account an average advancement of approx. 1.5 concrete lifts per week and the simultaneous concreting of two blocks, the completion of the dam will need approx. 10 to 12 weeks.

Considering the limited height of the dam it is generally planned to complete an entire block before concreting the next one.

6.5 *Monitoring of the existing dam during works*

During the construction works, and in particular during the excavation phase for the new foundation, a specific monitoring program is adopted to the existing dam.

Movements are measured using three inclinometer installations on the dam crest and a temporary pendulum installed in the central zone of the spillway (Fig. 14). In addition, every two weeks, geodetic measurements are carried out. After almost two months of work and at the end of the excavations, no particular problems related to the behaviour of the existing dam were observed.

Figure 14. Overview of the old and the new dams during construction.

7 FINAL CONSIDERATIONS

Following expansion of the concrete affected by a ASR, irreversible displacements of the existing arch das led to a state of cracking spread out over the whole dam. By a further advancement of the concrete expansion the safety of the dam could not be longer assured.

Taking the small dimension of the structure into account, the reconstruction of new dam was considered the most advantageous solution instead of the rehabilitation of the existing one.

This solution also permitted to increase the hydraulic safety of the dam by designing a new spillway with increased capacity and providing a more efficient bottom outlet.

The risks, which are accompanied by construction of the new dam at the downstream toe of the still operating old dam could easily be managed through an appropriate work program, a restricted reservoir management during works as well as appropriate construction methods especially during excavation of the foundation.

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La adecuación de la presa de Alarcón

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RESUMEN: El Embalse de Alarcón, con una capacidad superior a un kilómetro cúbico, está situado en el río Júcar y representa la infraestructura principal de regulación de los recursos de la cuenca de dicho río. En 2005se redactó el "Proyecto de Adecuación a La Normativa Vigente del aliviadero de la Presa de Alarcón", en el que se afrontaba la modernización del conjunto de elementos hidráulicos. Las actuaciones previstas debían ejecutarse bajo la premisa de una mínima afección a la explotación ordinaria del mismo y sin menoscabo de los más estrictos criterios de seguridad. Destaca el rediseño del aliviadero de superficie.

Desde el punto de vista estructural, se analiza el contacto del hormigón existente con el nuevo, diseñando una red de drenaje que limita la generación de subpresiones en la superficie de contacto y el bulonado de unión entre ambos. Las obras están actualmente en ejecución.

ABSTRACT: Alarcón reservoir, with a capacity exceeding one cubic kilometer, is located on the river Júcar and represents the main regulatory infrastructure resources of the river basin. In 2005, the adaptation project of the Alarcón dam spillway was made. This project includes the modernization of all hydraulic components of the dam. The planned measures in the Project must be implemented under the premise of a minimal condition for normal operation thereof and without prejudice to the highest standards of safety. Stands out the redesign of the spillway.

Structurally, the contact between the existing concrete and the new has been analyzed, so it has been necessary to design a drainage system that limits the generation of uplift on the surface of contact and the bolting of union in-between both. The works are currently underway.

1 INTRODUCCIÓN

1.1 *Antecedentes*

La presa de Alarcón, situada en el cauce del río Júcar en el TM. de Alarcón (Cuenca) fue terminada y puesta en explotación en el año 1955.

El Embalse de Alarcón, con una capacidad superior a un kilómetro cúbico, está situado en el río Júcar y representa la infraestructura principal de regulación de los recursos hídricos de la cuenca de dicho río, que se extiende entre las Comunidades Españolas de Castilla La Mancha y Valenciana. Por otra parte, y debido a su ubicación estratégica entre las cuencas del Tajo y del Segura, contribuye de manera decisiva a la explotación del acueducto Tajo—Segura, influyendo de manera notable en el desarrollo de la Comunidad de Murcia. Por todo ello, se puede considerar como uno de los embalses de mayor relevancia de España.

Responde a la tipología de presa vertedero de gravedad con planta recta, presentando un notable valor patrimonial. Las instalaciones han permanecido en servicio ininterrumpidamente desde el final de su construcción.

Durante 1994 se llevó a cabo una reparación localizada en los canales de descarga del aliviadero, motivada por el estado de deterioro de su superficie. Esta reparación consistió en la limpieza y saneado de su superficie, imprimación con resina epoxi dejando ventanas de drenaje, colocación de una malla electrosoldada y la proyección final de un mortero aditivado.

Con motivo de la suelta de agua por el aliviadero de la presa de Alarcón acaecida durante la primera quincena de Mayo de 1998, se produjo el levantamiento de diversos paños de su superficie, con el deterioro progresivo de la misma según continuaba dicha suelta, afectando al hormigón de base, sobre todo en la zona de la estructura de restitución al cauce, situada al pie de la presa.

Figura 1. Vista de los 3 vanos desde aguas abajo.

Figura 2. Estructura de restitución del aliviadero.

Se adjuntan fotografías del estado del aliviadero, que ilustran lo anterior.

En el año 2004 se redactaron la Primera Revisión y Análisis de la Seguridad, las Normas de Explotación y el Plan de Emergencia.

El PROYECTO DE ADECUACIÓN DEL ALIVIADERO DE LA PRESA DE ALARCÓN A LA NORMATIVA VIGENTE se adjudicó a la UTE constituida por las Empresas Consultoras "Jesús Granell, Ingenieros Consultores y MS Ingenieros S.L" y se redactó en 2005. Incluía la reparación del aliviadero, la modernización del conjunto de elementos hidráulicos de la presa y otras actuaciones necesarias que se describen en el presente artículo.

Las obras se adjudicaron a ALPI en 2008 y se están ejecutando en la actualidad, con fecha de finalización en 2011.

Por otro lado, en el año 2006 finalizaron las Obras de Sustitución y Adecuación de la Toma de Agua de la Presa de Alarcón.

1.2 *Características principales de la presa de Alarcón*

A continuación se detallan las características principales de la presa de Alarcón en la actualidad:

EMBALSE Y PRESA

- Superficie de la cuenca: 2938 Km^2
- Capacidad del embalse: 1117.826 Hm^3
- Caudal punta avenida de 1000 años: 1975 m^3/s
- Caudal punta avenida de 5000 años: 2406 m^3/s
- Caudal punta avenida de 10000 años: 2916 m^3/s
- Tipo: gravedad planta recta
- Altura sobre cimientos: 70 m
- Cota de coronación: 814.00
- Cota de máx. nivel normal: 806.00

ALIVIADERO

- Tipología: 3 vanos de 15 m
- Compuertas: TAINTOR de 7 m de altura
- Cota de apoyo de compuertas: 799.00
- Cota superior de compuertas: 806.00
- Capacidad act. aliviadero (1000 a): 1429.6 m^3/s
- Capacidad act. aliviadero (5000 a): 1513.5 m^3/s
- Capacidad act. aliviadero (10000 a): 1645.4 m^3/s

DESAGÜES DE FONDO

- Capacidad a N.M.N.: 78.6 m^3/s

TOMA DE AGUA

- Capacidad a N.M.N.: 114.0 m^3/s

1.3 *Actuaciones a realizar en la presa de Alarcón*

Las obras proyectadas obedecen a las conclusiones de los documentos citados en los antecedentes, para la adecuación a la Normativa vigente.

La característica más destacable de estas obras es que se trata de obras de remodelación de estructuras hidráulicas en funcionamiento, por lo que deben ejecutarse interfiriendo mínimamente en la explotación. Y por supuesto, con los más estrictos criterios de seguridad en la ejecución.

Por otro lado, se ha respetado la estética global de la presa de Alarcón.

Se describen a continuación las obras proyectadas.

2 ADECUACIÓN DEL ALIVIADERO

Los ensayos hidráulicos en modelo reducido realizados en el CEDEX, pusieron de manifiesto la necesidad de acometer la remodelación de las dos pilas laterales del aliviadero en su parte de aguas arriba. En efecto, en la actualidad, estas pilas presentan una forma poco hidrodinámica y son excesivamente cortas para proporcionar una adecuada alimentación.

Así, en los ensayos se constata la presencia de fuertes depresiones que se localizan junto a estas pilas laterales en las zonas de empotramiento de los vertederos. Estas depresiones, debidas

indudablemente a la presencia de flujo lateral en la alimentación, están provocadas por la falta de la adecuada canalización del flujo al no tener la longitud adecuada ambas pilas laterales.

Para eliminar este defecto en el aliviadero, se ha proyectado la remodelación de las partes de aguas arriba de ambas pilas laterales. Consta, esta remodelación, de las siguientes operaciones:

- Picado y descabezado de la puntera actual de las pilas laterales, sin empleo de explosivos.
- Preparación de las superficies de unión, con el fin de eliminar todo el árido que quede suelto o mal embebido en el resto de la masa de hormigón.
- Ejecución de los anclajes de unión.
- Encofrado y hormigonado de las pilas con su nueva forma.

Además, la superficie del paramento, en la zona del escarpe, no responde a un perfil hidráulico adecuado, tipo Creager, como se muestra en la sección tipo que se adjunta.

Esta circunstancia ha dado origen a fenómenos de depresión y cavitación en la solera de los canales de descarga, contribuyendo a acelerar su deterioro.

Las nuevas formas han sido incorporadas al ensayo hidráulico en modelo reducido, pudiéndose apreciar la desaparición de las depresiones y el logro de la correcta alimentación (para caudales del orden de 1700 m^3/s).

2.1 *Demoliciones*

Para proceder a la remodelación de los tres vanos del aliviadero, se ha previsto en primer lugar, la demolición del actual hormigón superficial como mínimo en 1 m de espesor, dejando la superficie demolida bien perfilada y en forma escalonada. Esta forma escalonada, al establecer una serie de plataformas horizontales, facilitará las posteriores operaciones de:

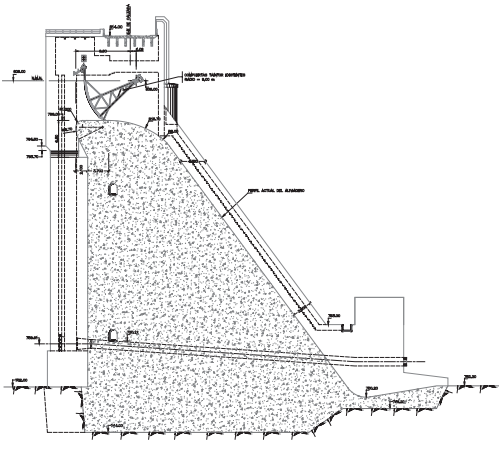

Figura 3. Sección tipo por el aliviadero existente.

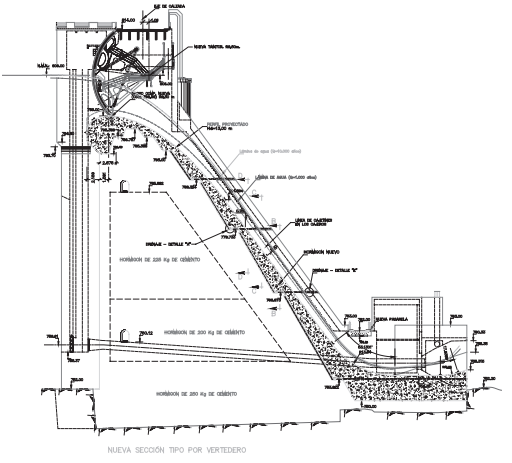

Figura 4. Sección tipo por el aliviadero tras su adecuación.

- Limpieza y tratamiento superficial de la superficie demolida, con la completa eliminación de cualquier elemento o árido suelto y sellado de cualquier fisura.
- Ejecución de la red de zanjas de drenaje.
- Ejecución de toda la red de bulonaje.
- Ferrallado de la nueva solera.
- Encofrado y hormigonado por tongadas de la nueva solera.

Todas las demoliciones se harán sin empleo de explosivos, por procedimientos manuales utilizando martillos, rozadoras o cortadoras que no produzcan vibraciones dañinas en toda la estructura del cuerpo de presa.

La geometría final de las superficies demolidas se muestra a continuación en una sección transversal por el eje de vano del aliviadero.

2.2 *Revestimientos*

Todas las superficies y zonas demolidas, se revestirán posteriormente con hormigón armado HA-25/P/35/IIa de baja retracción para conformar la mejora estructural y de comportamiento hidráulico del perfil del aliviadero.

El nuevo perfil del escarpe del aliviadero se ha diseñado de acuerdo con las recomendaciones que se recogen en "Hydraulics Design Criteria", publicado por el U.S. Corps of Engineers. Así, su perfil transversal responde a la siguiente ecuación:

$$X^{1.85} = 2 \times Y \times H_d^{0.85} \quad (1)$$

donde H_d es la carga o lámina de diseño sobre la cresta, de valor igual a 13 metros.

En los casos de desagüe bajo compuerta, se han considerado distintos grados de apertura de las compuertas.

Con estos cálculos, se ha verificado que la embocadura se encuentra adecuadamente diseñada y que el aliviadero es capaz de desaguar la avenida de 10000 años de periodo de retorno con funcionamiento en lámina libre.

2.3 *Drenaje*

Para controlar y reducir al máximo las presiones intersticiales en el contacto entre hormigones, se ha proyectado una red de drenaje consistente en: cuatro peines en cada uno de los canales de aliviadero, situados sobre la superficie demolida.

Cada peine esta constituido por cinco zanjas a lo largo del paramento y una horizontal situada en la parte inferior de cada peine que las colecta. En el interior de las zanjas, que poseen unas dimensiones de 0.35 m de profundidad y 0.20 m de espesor en la base, con taludes de 1H:2V, se dispone una tubería de hormigón poroso de 15 cm de diámetro, que es la encargada de la recogida de las filtraciones. Una vez montadas las tuberías, se rellenará toda la sección de las zanjas con hormigón poroso.

Los drenes horizontales, se empotran en los cajeros de los canales de descarga, donde desembocan en una tubería colectora de fibrocemento de 15 cm de diámetro; esta tubería, siguiendo esta misma cota por el cajero, evacua las posibles filtraciones fuera del nuevo revestimiento a través de los citados cajeros.

2.4 *Bulonado*

En toda la zona de aliviadero que ha sido picada para poder construir el nuevo revestimiento, se realiza una cuadrícula de bulones que servirán para unión de ambos hormigones y para absorber las tensiones que se produzcan en el contacto. Los anclajes se definen son de diámetro ϕ 25 y la cuadrícula es variable entre 0.9 y 1.2 m. Se adjunta una planta donde se observa la distribución de los anclajes y las zanjas de drenaje.

2.5 *Sustitución de las compuertas Taintor existentes*

Dentro de las actuaciones previstas para la rehabilitación del aliviadero de la presa de Alarcón, figura como una de las más relevantes la sustitución de las compuertas Taintor existentes.

Como consecuencia de conservar el eje de giro de las compuertas y de haber proyectado un nuevo perfil Bradley, el radio de giro de las nuevas

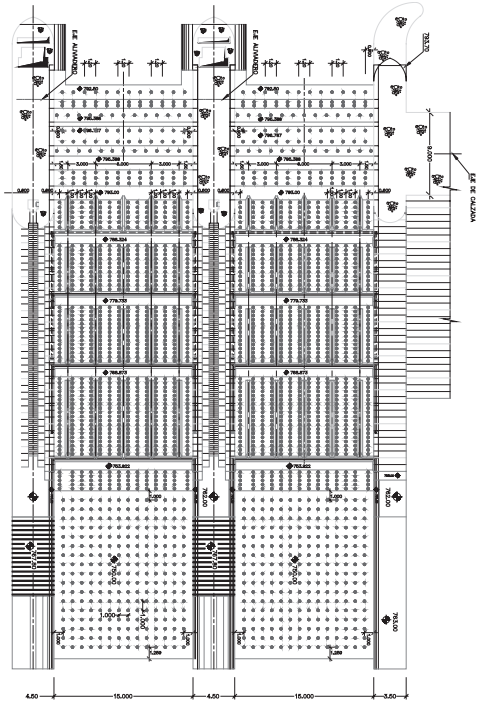

Figura 5. Planta del aliviadero. Drenaje y bulonado.

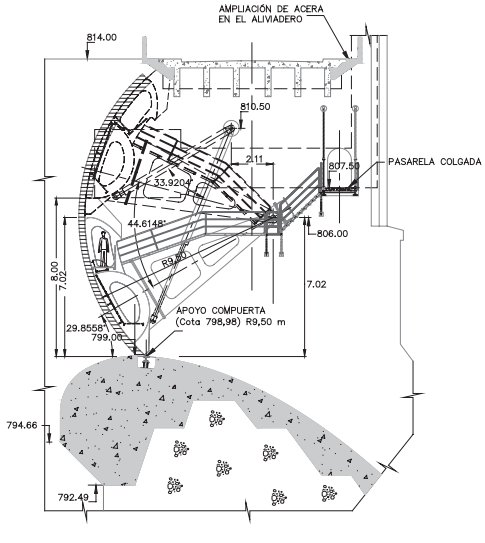

Figura 6. Escarpe del aliviadero y nueva Taintor.

compuertas es de 9.50 m. Las nuevas compuertas se han diseñado y calculado utilizando las técnicas más actualizadas, dotándolas de secciones metálicas cerradas en cajón y de un sistema de accionamiento oleohidráulico mediante servomotores.

Se adjunta un esquema del nuevo escarpe y de la compuerta Taintor.

2.6 *Estructuras terminales*

La estructura de lanzamiento proyectada se define geométricamente, para cada vano, mediante la intersección de dos planos laterales con una solera de perfil cilíndrico de eje horizontal y un plano paralelo al eje longitudinal de la presa, situado este último en prolongación de la solera cilíndrica por aguas abajo.

Los dos planos laterales, simétricos respecto del eje de cada vano del aliviadero, estrechan los vanos de 15 a 5 m en una longitud de cerca de 12.50 m. La solera cilíndrica, que se sitúa centrada sobre el vano, es tangente al plano horizontal en su extremo de aguas arriba, tiene un radio de 10 m y un ángulo de lanzamiento de 17.84°. El plano paralelo al eje longitudinal tiene una longitud de 1.58 m.

Se muestra a continuación una planta y un alzado frontal.

El alcance máximo del chorro, medido a partir de los trampolines, es de 40 m.

Para poder construir estas estructuras es necesario ejecutar una contraataguía provisional de unos 3.0 m de altura, de material todo uno, ubicada aguas abajo del actual cuenco de recepción.

La sección tipo de la contraataguía posee una anchura en coronación de 5.0 m y taludes simétricos de 2H:1V.

La construcción de la contraataguía permite dejar en seco el cuenco durante los 18 meses necesarios para remodelar los desagües de fondo y los trampolines del aliviadero. Durante este intervalo de tiempo se podrá investigar el estado de la roca en el cuenco y proceder a su tratamiento si se estima oportuno.

A continuación se adjunta una fotografía, en la que se puede observar la demolición escalonada y los anclajes dispuestos.

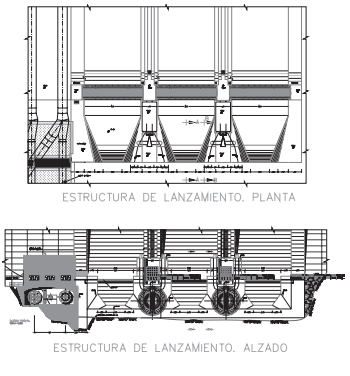

Figura 7. Estructuras terminales del aliviadero.

Figura 8. Vista de las obras del aliviadero.

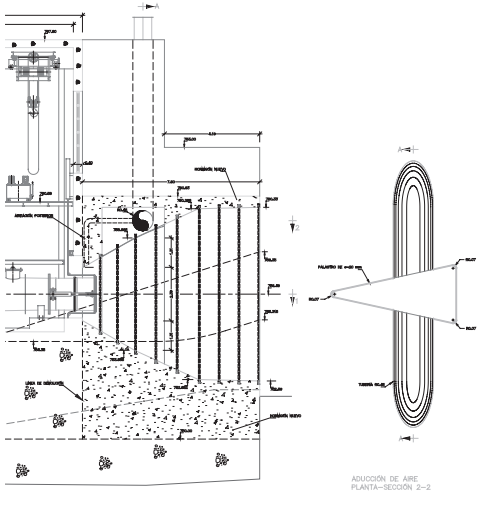

Figura 9. Esquema de la válvula de los desagües de fondo.

3 REMODELACIÓN DE LA SALIDA DE LOS DESAGÜES DE FONDO

La disposición actual de las válvulas HOWELL-BUNGER de los desagües de fondo proporciona un excelente funcionamiento hidráulico al conjunto de este órgano de desagüe, si bien la apertura de los chorros a 45° origina una neblina de proporciones excesivas, la cual combinada con las extremadas temperaturas de la zona en invierno, da lugar a la formación de hielo sobre las especies arbóreas existentes en la ladera izquierda, aguas abajo de la presa.

Para suprimir este efecto pernicioso, se ha previsto la colocación de sendas campanas deflectoras a la salida de las válvulas, que sin reducir el efecto disipador de energía que proporcionan éstas, concentran los chorros hacia una forma más

cilíndrica, evitándose de esta manera la formación de la extensa neblina.

Se adjunta un esquema de la válvula.

Las campanas se construyen en palastro de acero al carbono de 20 mm de espesor, y van dotadas de los correspondientes rigidizadores así como de la debida protección anticorrosión. Quedan embebidas en el hormigón armado que conforma los bloques de las estructuras terminales-trampolines del aliviadero.

4 REPARACIÓN DEL PARAMENTO INCLINADO DE AGUAS ABAJO DE LA PRESA

Para la reparación de este paramento, se analizaron diversas alternativas, desde un tratamiento superficial a base de sellado y proyección de pinturas epoxídicas aplicado con pistola, hasta el más completo y sofisticado recubrimiento con losas de hormigón armado, in situ o prefabricado.

Finalmente, dada la importancia y envergadura de la Presa de Alarcón, se ha elegido la segunda opción, que aunque más cara proporcionará a la presa un aspecto de muy buena terminación, a la vez que un paramento de gran durabilidad.

Se ha proyectado, por consiguiente, un revestimiento fabricado in situ y formado por un mosaico de losas independientes de hormigón armado de 30 cm de espesor, ancladas al cuerpo de presa mediante 5 anclajes ϕ 20 por losa y con una longitud de 2.0 m (embebidos en el hormigón actual 1.75 m).

Las dimensiones de las losas son variables entre 4,0 m y 5,23 m de anchura y 1,83 y 2,17 m de altura.

Para la modulación de los paramentos mediante las losas citadas, se han respetado las juntas entre bloques de la presa. Asimismo, se han previsto entre losas adyacentes, unos cajeados para drenar el contacto con los paramentos de la presa y eliminar así las subpresiones en dicho contacto.

Se muestra a continuación una fotografía, donde se observa, en la parte inferior, el paramento de aguas abajo inclinado ya reparado y en la parte superior el paramento vertical de la presa.

Figura 10. Vista del paramento de aguas abajo.

5 ACTUACIONES AMBIENTALES

Las actuaciones de restauración ambiental y paisajística previstas son:

- Captura y traslado de peces atrapados en el cuenco amortiguador, entre la contraataguía y el pie de presa.
- Colocación de escollera en el punto de vertido de las tomas.
- Regeneración de las especies arbóreas de la ladera izquierda, aguas abajo de la presa mediante el tratamiento selvícola de la zona (corta de pies muertos) y la plantación de pinos (*Pinus halepensis*), encina (*Quercus rotundifolia*) y sabina negra (*Juniperus phoenicea*).
- Restauración del vertedero de residuos inertes con plantación de pino carrasco (*Pinus halepensis*), encina (*Quercus rotundifolia*), romero (*Rosmarinus officinalis*) y espliego (*Lavandula latifolia*).
- Integración paisajística mediante la retirada del material de la contraataguía y de la plataforma de acceso a ésta y mediante la creación de un nuevo mirador en las proximidades de la nave taller.

6 OTRAS ACTUACIONES

Existen otras obras complementarias de mejora de las instalaciones de la presa.

6.1 *Aceras en coronación*

En coronación se han proyectado unas piezas especiales de 1.50 m de altura y anchura variable, lo que proporciona un sobreancho de aceras de 0.77 m. Se colocará una barandilla nueva en toda la coronación y se mejorará el alumbrado.

Está prevista la rehabilitación de las vigas del puente sobre el aliviadero así como la ampliación de la anchura del mismo tanto aguas arriba como aguas abajo.

6.2 *Pasarelas en vanos del aliviadero, aguas abajo de las TAINTOR*

En cada uno de los tres vanos del aliviadero y a la cota 807.50, se proyectan sendas pasarelas colgadas de 15 m de luz y 1.70 m de ancho.

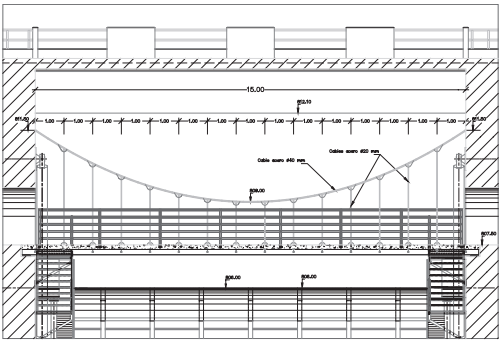

Figura 11. Pasarela en vanos del aliviadero.

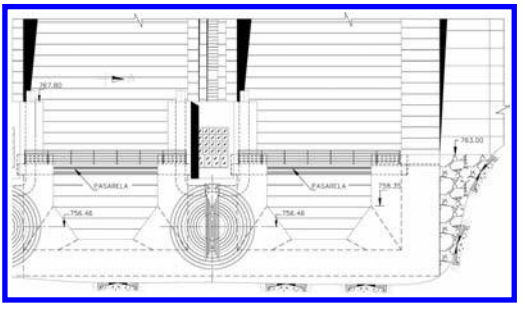

Figura 12. Alzado de las pasarelas de acceso a la cámara de los desagües de fondo.

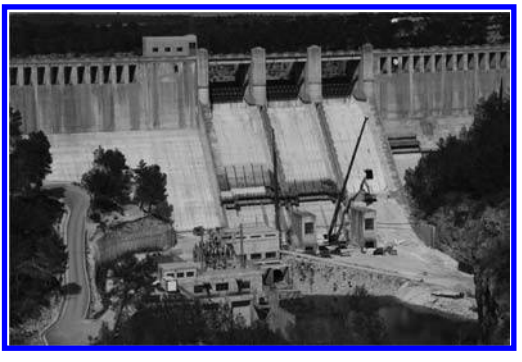

Figura 13. Vista general de las obras.

El tablero está formado por 2 perfiles metálicos de sección IPE-200 separados entre ejes una distancia de 1.60 m, sobre los que apoya una losa de hormigón armado de 15 cm de espesor y 1.70 m de anchura. Los perfiles principales se encuentran arriostrados horizontalmente mediante 16 perfiles, también metálicos, de sección IPE-100.

La pasarela cuelga de dos cables de directriz parabólica de sección circular de 40 mm de diámetro a través de 28 cables verticales (péndolas) de sección circular de 20 mm de diámetro. Los 28 cables se reparten en dos filas de 14 cables, una a cada lado de la pasarela coincidiendo su alineación con las directrices de las dos vigas metálicas del tablero y naciendo de la intersección de las vigas-riostra interiores con las dos vigas principales, siendo por lo tanto, la separación entre cables verticales, y por lo tanto de las vigas-riostra, de 1 m en dirección longitudinal a la pasarela.

Las dos vigas metálicas del tablero apoyan vertical y lateralmente en las pilas del aliviadero mediante los correspondientes apoyos de neopreno. El apoyo lateral se diseña con objeto de absorber las reacciones horizontales producidas por la carga de viento.

Los cables parabólicos anclan en sus extremos en las pilas del aliviadero, siendo la distancia en vertical desde la cara superior del tablero de la pasarela hasta el anclaje de los mismos de 4 m, y la distancia, también en vertical entre la cara superior del tablero y el vértice de la parábola (en centro de vano), de 1.50 m. Finalmente se disponen barandillas metálicas a ambos lados del tablero, de 1.25 m de altura. Se adjunta el alzado de la pasarela.

6.3 *Pasarelas para acceso a las cámaras de los desagües de fondo*

Las actuales pasarelas que dan acceso a las cámaras inferiores de los desagües de fondo poseen sección insuficiente para facilitar el acceso de vehículos. En consecuencia se han rediseñado, dotándolas de una sección transversal adecuada para tal fin, a la vez que su rasante se ha adaptado a la nueva geometría general de la plataforma de acceso a la nueva cámara de las tomas de agua.

Se adjunta un alzado, donde también se puede observar la válvula de los desagües de fondo.

6.4 *Cámaras y Galerías*

Se ha previsto la adecuación y mejora de la funcionalidad de cámaras de las compuertas TAINTOR y galerías de acceso a las mismas. Y la adecuación del alumbrado en galerías y cámaras.

6.5 *Alumbrado y red de fuerza*

Se ha previsto la Adecuación de la red de fuerza y alumbrado a la Normativa vigente y la mejora del alumbrado en el camino a las oficinas.

6.6 *Accesos*

Se incluye el ensanche y afirmado del camino de acceso a los almacenes desde el estribo izquierdo y la adecuación de la escalera de acceso a pie de presa desde el estribo izquierdo.

7 CONCLUSIONES

El Embalse de Alarcón, con una capacidad superior a un kilómetro cúbico, está situado sobre el río Júcar y representa la infraestructura principal de regulación de los recursos hídricos de la cuenca de dicho río.

Con motivo de la suelta de agua por el aliviadero de la presa acaecida durante la primera quincena de Mayo de 1998, se produjo el levantamiento de diversos paños de su superficie, con el deterioro progresivo de la misma según continuaba dicha suelta, afectando al hormigón de base, sobre todo en la zona de la estructura de restitución al cauce, situada al pie de la presa.

A lo largo del año 2005, se redactó el "Proyecto de Adecuación a La Normativa Vigente del aliviadero de la Presa de Alarcón", en el que se afronta la modernización del conjunto de elementos hidráulicos de la presa, de conformidad con el vigente Reglamento sobre Seguridad de Presas y Embalses.

Las actuaciones previstas en el proyecto, pese a su complejidad y debido a la importancia estratégica del embalse, han de ser llevadas a cabo bajo la premisa de una mínima afección a la explotación ordinaria del mismo y sin menoscabo de los más estrictos criterios de seguridad durante la ejecución de las obras. Destaca, dentro de este conjunto de actuaciones, el rediseño del aliviadero de superficie—para un caudal de 1700 m^3/s—, la sustitución de sus tres compuertas TAINTOR, la remodelación de los canales de descarga y de la estructura de disipación de energía al pié de presa. También se prevé la adecuación de los desagües de fondo, la restauración del paramento inclinado de aguas abajo y otras obras accesorias.

Desde el punto de vista estructural, se analiza escrupulosamente el contacto del hormigón del cuerpo de presa existente con el nuevo, diseñando una red de drenaje que controla y limita la generación de subpresiones en la superficie de contacto y el bulonado de unión entre ambos. Las obras están actualmente en ejecución y una vez finalizadas, el Embalse de Alarcón se encontrará en disposición de afrontar otros 50 años de explotación ininterrumpida.

Se muestra a continuación una fotografía con el estado general de las obras.

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Cement-bentonite cut-off wall as remedial water proofing of Barriere Dam, Cabonga Reservoir, Canada

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ABSTRACT: This paper describes the technical pre-project study, engineering, design, and construction of the water tightness remedial works at the Barriere Dam located northwest of Montreal, in the province of Quebec, Canada. The maximum height of the dam and the hydraulic head are about 8 and 4,5 m respectively. The original dam was a wood crib structure built on an overburden foundation. Later at two different times, the dam was modified. Furthermore, a regulation structure was added to discharge towards another reservoir. The new waterproof barrier was to be a self-hardening cement-bentonite slurry cut off wall in the existing embankment. In addition, to assure the permanence of the work, a stabilizing upstream rockfill berm was placed above the impervious blanket that is susceptible to liquefaction after the maximum design earthquake.

1 INTRODUCTION

1.1 *Description of Cabonga Reservoir*

The Cabonga Reservoir is located in northeastern Canada, precisely northwest of Montreal in the province of Quebec (see figure 1). It is a component of the Ottawa River Watershed (see figure 2). Its exploitation is optimized to meet needs of: hydro-electricity, protection against floods, and recreation.

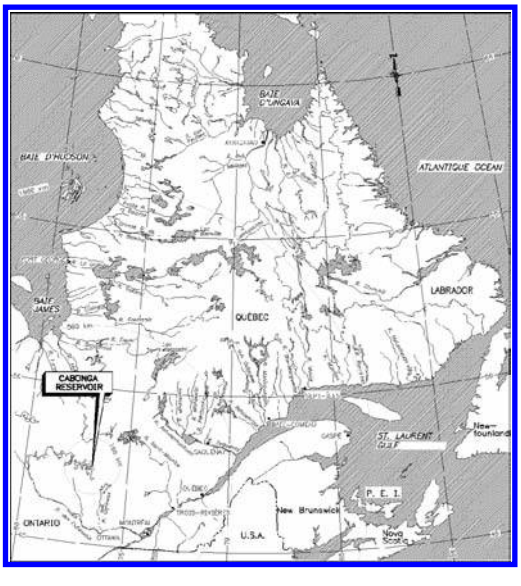

Figure 1. Cabonga Reservoir localisation.

This reservoir is contained by ten (10) retaining structures of which two (2) are concrete structures, that assure reservoir perimeter closure, retain water, control the water level and permit discharges towards downstream. The Cabonga Reservoir area is 434 km^2 and it has a useful reserve of 1 511 hm^3.

The reservoir south rim structures are Cabonga Dam and Baie-des-Seize dyke. The north ones are Barriere Dam, Baie-des-Sept-Milles and Wagoose dykes. The discharge system is composed of the Cabonga Dam in the south, which allows the water supply of the Baskatong Reservoir via the Gens de Terre River, and the Barriere Dam in the north, which supplies the Dozois Reservoir.

The Barriere Dam is located in La Verendrye wildlife reserve, 470 km northwest of Montreal (by road), in the province of Quebec. It has an overall

Figure 2. Ottawa River Watershed.

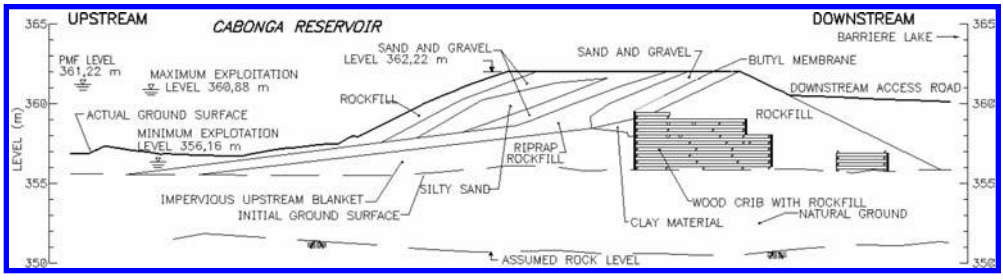

Figure 3. Cross-section after 1994 emergency remedial works.

length of 244 m, a maximum height of about 8,0 m and a maximum hydraulic head of approximately 4,5 m. The dam is composed of three segments: the right embankment which constitutes a large portion of the original dam, the concrete regulation structure, and the left embankment which is the fill between original ground and the excavation for the regulation structure construction.

1.2 *Barriere Dam brief history*

At the time of commissioning, in 1928–1929, the dam had a wood crib structure with rockfill, built on an overburden foundation. Its water tightness was assured by an upstream facing composed of a double row of wood boards on the upstream face of the wood crib and by an upstream lightly inclined blanket of low permeability material, about 38 m long at the highest section of the dam. In 1966, the dam was temporarily modified to an embankment of rockfill, sand and gravel, built after it had been levelled off at 3,35 m below the top of the original wood crib. A butyl membrane anchored to the top of the remaining upstream wood facing and covering the new earthfill (between sand and gravel layers) was applied for water tightness. Finally, this temporary work became permanent.

In 1975, the Barriere regulation structure was built and the wood crib section in the foot print of the regulation structure was removed. The remaining wood crib and the butyl membrane did not reach the regulation structure according to the information available on construction issued drawings. Afterward, that space was filled with an impervious material.

In the spring of 1994, following observations of large additional water infiltrations, downstream resurgences and settlement on the crest of the right embankment, it was considered that the dam required emergency remedial works. In order to mitigate infiltrations, silty sand material was dumped in water on the upstream face of the existing dam, between two layers of the sand and gravel filter. The exposed upstream face was covered with rockfill. In addition, the downstream area was raised about 1 to 1,5 m with gravel and sand fill. Figure 3 shows this cross-section after emergency remedial works. Table 1 shows the properties of the right embankment.

Table 1. Properties of right embankment of Barriere Dam.

Crest level (geodesic) (m)	362,22 min.
Maximum height (m)	8±
Crest length (m)	244
Crest width (m)	14±
Mean upstream slope	2,5 H:1 V
Mean downstream slope	2 to 3 H:1 V
Maximum hydraulic head (m)	4,5±
Freeboard (m)	1,34
Upstream impervious blanket length (m)	38±[1]

Note: [1] From the upstream face wood crib at the deepest location.

2 TECHNICAL PRE-PROJECT STUDY FOR REHABILITATION

This section describes the stages of the pre-project study.

2.1 *Description of the problem, the fill and the natural foundation*

2.1.1 *The problem*

In the summer of 2005, as the owner had observed continued infiltrations, a study was undertaken to find a solution. It began with a documentation review with the emphasis on two (2) of them that were written after the emergency remedial works of 1994: (1) The special behavior report, and (2) The soil investigations report. In the special behavior report, there was a detailed synthesis and historic of the right embankment. Furthermore, with the results of the 1994 investigations, the typical

cross-section of this embankment was deducted at boreholes locations along the dam.

During proposal preparation, three solutions were proposed: a Cut Off Wall (COW), a new dyke downstream and a downstream berm that would ensure the dam stability. The owner decided to explore the COW concept and proposed a sheet pile cut off. First of all, three (3) different types of COW were studied: (1) A steel sheet pile cut off, (2) A self-hardening cement-bentonite slurry wall, and (3) A jet grout wall. During the autumn of 2005, these COWs necessitated additional field investigations that were done near the presumed axis location (near upstream crest). The objectives of these surveys were to validate the internal geometry (various zones) of the right embankment, the nature and density of the fill materials and foundation, and the characteristics and depth of the bedrock.

2.1.2 *The fill*

The fill thickness from the crest to the natural foundation varies from 1,0 to 6,6 m. These fill materials mainly consist of (see figure 3):

- fill placed during emergency remedial works of 1994: upstream rockfill protection, transition (sand and gravel), new core (silty sand with small amount of gravel);
- the 1966 riprap rockfill;
- a soft layer, composed of gravelly sand or sand and gravel, some silt to silty and gravelly sand, sometimes with some organic materials, from 1,3 to 2,9 m of thickness is present above the natural foundation (this layer is probably the upstream impervious blanket).

The 1994 dumped fill is loose. For this fill, ranging between a depth of 0 to 3,6 m, the N blowcount for the Standard Penetration Test (SPT) varies from 5 to 13 (excluding some higher values) for an average value of approximately 8. Figure 4 shows the SPT blowcount for borehole materials that would be intersected by COW axis. In this figure the refusal line corresponds to refusal depth for a specific split spoon. Note that the N blowcount of 1994 and 2005 boreholes had not always been measured with the standard "B" size split spoon. Many of the N values had been obtained with "N" while some others with "H" size split spoon but here the results presented are all converted to "B" size with correlations found in the literature (Winterkorn 1975, Hunt 1984). Then, for a depth varying from ±3,8 to 6,1 m, the soils were also described as loose. This interval should correspond to the upstream impervious blanket with N values from 2 to 26 (2 to 10 for 2005 borings). The average blowcount is approximately 12 but if the 2005 borings are only considered the average is 5.

2.1.3 *The natural foundation*

The investigations synthesis of all borings showed that the natural foundation, under the fill, consists of heterogeneous soils with variable size grading. The natural ground thickness under the foot print of the dam varies from 1,2 to 5,9 m. Generally, the foundation is composed of gravel and sand to gravelly sand, with a small amount to some silt. The natural ground is characterized by density varying from compact to very dense.

The SPT blowcount of the natural foundation ranges from 6 to 48. The average N value is approximately 26 excluding all values converted to "B" size that give values higher than 30 (data with value 60 on figure 4). Moreover, several cobbles and boulders horizons were localized in boreholes (see right section of figure 4 for locations). During investigations, core drilling was required to pass through these layers. Finally, it was then reasonable to believe that the COW would have a maximum depth of about 12 m from the crest to bedrock level and an area of approximately 1 420 m^2.

2.2 *Description of the three potential COW*

This section describes the three types of COW that were briefly studied to determine the optimal alternative for continuation of technical pre-project study. The choice would be done in relation to construction cost, environment and certificate of approval request from government.

2.2.1 *Sheet pile cut off*

This alternative consists of driving sheet piles into the fill and foundation to bedrock. Here, it would be placed at the upstream crest edge because it would not be desirable to drive the sheet pile in the 1966 riprap zone intercepted with boreholes close to the upstream crest edge (see figure 3). The driving of the sheet pile would require the placement of a temporary random rockfill platform which would cover partly the upstream slope of the right embankment or the excavation of a work platform in the 1994 fill.

2.2.2 *Self-hardening cement-bentonite slurry*

This second alternative is a wall built in one phase (slurry maintains the trench open during excavation and hardens to become the COW). The axis location had to be near the upstream crest edge. For this work, prior to the realization of the COW, a gravity injection would be required to fill the voids between the stones of the 1966 riprap. This injection would be necessary to avoid loss slurry which could cause a collapse of the cut off trench. A work platform of about 5 m wide would be required to use a shovel excavator with a long reach boom. Due to the axis location of the COW, the work platform

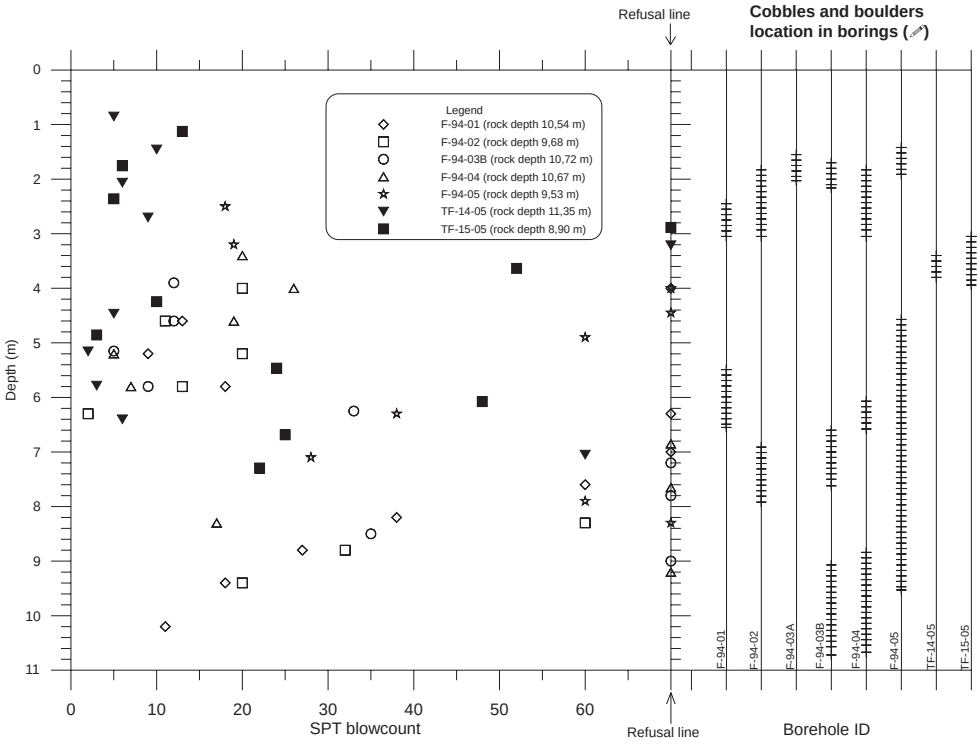

Figure 4. SPT blowcount in planned axis (technical pre-project study).

would be done with excavation in the 1994 fill and an upstream protection would be necessary to make sure that the cement-bentonite grout would not contaminate the reservoir in case of incident.

2.2.3 *Jet grouting curtain wall*

This technique of soil improvement consists in eroding and mixing the *in situ* soil with water cement grout. The grout mix is jetted, with the aid of special tools, at high speed created by high pressure. The column spacing is chosen based on *in situ* soil to be sure that the overlap of adjacent columns maintains the minimal width of the COW. At the Barriere Dam, the position of the curtain wall would be the same as the previous alternative. The work platform would be the crest of the dam. It would be anticipated that a large amount of grout would be lost in the zone of the 1966 riprap. This method would also require the use of an upstream protection to not contaminate the reservoir.

2.2.4 *Financial analysis*

The financial analysis showed that sheet pile cut off and cement-bentonite COW had a similar estimated cost. The difference between them was about 20% with an accuracy of about ±30%. However, the jet grout was estimated 30% higher than the average cost of the two other methods.

2.2.5 *Choice for completion of technical pre-project study*

The technical team recommended to the owner the cement-bentonite COW because its cost was reasonable and Hydro-Quebec had experience for this type of work. Some doubts persisted for the driving of a sheet piles cut off as it was less expensive and easy to achieve. But, the technical disadvantages were: (1) Difficulty to drive sheet pile on the site because of the natural foundation (several zones of cobbles and boulders, and the high density with SPT blowcount up to 48); (2) Important risk of deviation due to presence of cobbles and boulders; (3) Possibility that the sheet pile cut off would not reach the bedrock; and (4) Impossibility to assure water tightness between sheet pile and bedrock without grout curtain. Moreover, if a problem occurred during sheet pile driving, extra work would be required. Additional costs would be anticipated and the impermeability of the new curtain would be difficult to estimate. The project management team decided to finish the technical pre-project study with cement-bentonite COW.

2.3 *Additional surveys to reduce uncertainty for the engineering phase*

During the technical pre-project study, some uncertainties were noticed. Those uncertainties concerned the bedrock level (particularly near the right abutment), the end point of the wood crib and the density of soils. Some informations found in the documentations suggested that the bedrock level is falling down in the right abutment region and two different locations were found for the end point of the wood crib (and butyl membrane) on the left side of the embankment. Moreover, the boreholes indicated that the dam is composed of low density soils.

In the autumn of 2006, topographic and geophysical surveys were done. The objectives of those surveys were to validate: the old topographic survey (made in an arbitrary level for which two conversion factors were found to transfer to geodesic level) and enlarge the covered area, the soils compactness (determination if superficial compaction was required to a depth of 1 m or 3 to 5 m for 1994 fill), the presence of depressions in the bedrock between the borings in the COW axis planned and the bedrock level in right abutment region.

The geophysical survey was made by a surface wave testing method named Modal Analysis of Surface Waves (MASW) (Karray 1999, Lefebvre et al. 2006, Karray et al. 2009 & 2010). The MASW method was developed at the Université de Sherbrooke, Canada, at the end of the nineties. MASW survey gives a shear waves velocity (V_s) profile for a specific interval related to sensor spacing and V_s is directly related to the rigidity of the material expressed as the shear modulus in the elastic range (G_{max}). Based on V_s, an indication of the soil density was obtained. This survey confirmed that the bedrock level is falling down in the right abutment region. The highest bedrock level is near the presumed end of wood crib. Also, MASW indicated that the rock surface has a poor quality. For some profiles, the data were difficult to interpret because of small difference in V_s between soils and poor rock.

As a last activity of technical pre-project study, a soils investigations campaign was carried out to validate the bedrock level at the right abutment based on MASW survey data. It was also to do test trenches in the planned COW axis, at the left abutment region, to ensure that the wood crib was not on this axis. The investigations showed that the bedrock level was fairly consistent with the MASW surveys and the wood crib was not in the planned COW axis. But large boulders (up to 1,4 m) were found in test trenches.

Due to these observations, the investigation was extended to identify borrow sources of impervious and granular materials because it was decided to build a till filled cut off trench at each abutment. A borrow source of till and another of granular (filter) materials were identified at a maximum overhead distance of 3 km. Table 2 summarizes all the investigations performed on the site.

2.4 *Conclusion of study*

The technical pre-project study was carried out between 2005 and 2007. The solution for the rehabilitation of the dam water tightness was a self-hardening cement-bentonite COW with an axis near the upstream crest of the existing dam. This axis was recommended because of the 1966 riprap, which was not removed before dumpimg the materials on the upstream dam in 1994. At the end of the study, an area potentially susceptible to liquefaction was identified between a depht of 4 to 6 m. No compaction has been proposed because that area is considered limited and MASW suggested no large region with low V_{S1} (normalized V_s for a vertical effective stress of 100 kPa (Youd & al. 2001)).

Table 2. Summary of investigations data.

Year	Borehole	Test pit	Test trench	Geophysical survey
1975	16[1]	–	–	–
1994	14[2]	8[3]	–	–
2005	5[4]	–	–	–
2006	–	–	–	330 m
2007	photo interpretation study (borrow site)			
2007	–	29[5]	4[6]	–
Total	35	37	4	330 m

Notes: [1]No data near planned axis, geological information for rock; [2]7 on crest, 4 at downstream toe, 3 at downstream area; [3]No test pits on crest; [4]Along planned axis of technical pre-project study; [5]28 for borrow sources, 1 at right embankment; [6]2 test trenches for wood crib location and 2 for *in situ* soils density.

3 ENGINEERING PHASE

This section describes the engineering process for the preparation of tender documents.

3.1 *Reconstitution of the typical cross-section*

During the engineering phase, the cross-sections (see figure 3) presented in the special behavior report done after 1994 emergency works was validated. Three cross-sections were selected to confirm the 1994 typical cross-section and to perform stability analysis: one corresponding to the highest section of the dam and the other two on each side.

As built drawings of the original construction in 1928–29 were used to locate the impervious upstream blanket on the plan view. Two versions of the as built drawings were found. The first one from the Gatineau Power Company, which was the first manager of the reservoir, was issued in March 1930. The other one, from the Quebec Stream Commission, was issued in September 1929. The shape of the blanket was different on those drawings. The longest one was used. Drawings issued for construction in 1975 for regulation structure were used to locate the end of the wood crib at the left abutment. It all helped to position by trial and error the upstream blanket.

The 1994 and 2005 boreholes confirmed the position of the upstream impervious blanket and it seemed that the longest one gave good results for all borings. In the location chosen, the level difference of the impervious blanket location was about 30 cm (more or less) for four boreholes, while for the others the difference was 10, 19 and 109 cm. The same exercise was done with the shorter blanket but it was not possible to get results as good as the first ones. A bathymetric survey was done in 2008 and finally confirmed these interpretations about the location of the upstream impervious blanket.

The 1994 topographic survey carried out during emergency works was found and used in relation to borehole results to deduce the internal geometry of the fill. The 2006 topographic survey was used as the base plan view. A manual sketch of 1966, found in the drawings management system of Hydro-Quebec, illustrated the repairs done in that year. This sketch was used to adjust the reconstructed cross-sections which did not agree well with the 1994 topographic survey. All these manipulations seem simple but it had to be managed with arbitrary levels in the imperial system (1928–1929) and in metric system (1994) with uncertain conversion factors to geodetic level.

3.2 *Seismic analysis*

At the time of soils properties review in preparation for the 30% engineering review a reasonable doubt persisted about the soil liquefaction potential in the event of the Maximum Design Earthquake (MDE) for the identified zone in the technical preproject study. The area coincided with the layer of upstream impervious blanket in the deepest portion of the natural river bed. It was therefore imperative to complete the analysis of this aspect for the 70% engineering review in order to achieve the design and estimate the mitigation alternatives costs. The liquefaction potential analysis was done with the empirical Seed method, presented in detail by Youd & al. (2001). The analysis was conducted using the results of corrected N SPT values obtained from 1994 and 2005 site investigations. Figure 5 shows the results in terms of safety factor versus depth.

The calculations with the empirical method of Seed indicated that there was a liquefaction risk for the upstream impervious blanket. Some areas in the natural foundation were also exposed to a liquefaction potential. However, these calculations do not confirm definitively the liquefaction potential. Considering the limitations of the simplified Seed method, the regional seismicity, and the severity of the seismic loading (MDE magnitude of 6,5), it was proposed to perform a dynamic study. The purpose of this study was, first, to confirm the liquefaction potential of soils and secondly, to define more precisely the affected area.

Several opinions were circulated about the necessity of dynamic study. A countervaluation was sought and the project was stopped a few months while the owner studied progress. It was finally decided to consider the upstream impervious blanket liquefiable and to design a berm that would ensure stability in the post-liquefaction condition. In addition, the axis of the COW was moved downstream of the upstream crest edge to be away from the critical rupture circle.

3.3 *Stability analysis*

The design of a dam must be such that its stability against sliding is ensured for all conditions (normal to extreme). In the case of Barriere Dam, the stability analyses were performed with reconstructed cross-sections described previously. The most important stability analyses for this structure were the construction stage and the post-liquefaction condition for the new upstream berm. The construction stage was important because a random rockfill would be dumped on a loose saturated till and it was considered that a static liquefaction flow failure may occur. The term static indicates that gravitational forces alone drive the failure movements (no seismic or dynamic forces). Accordingly, the berm was constructed in two stages. The first stage involves

the establishment of a pier and its axis coincides roughly with the rim of the upstream impervious blanket. The next step is filling the space between the downstream slope of the pier and the upstream slope of the existing dam to form the berm (see figure 6).

The stability analysis which represents the berm construction was modeled with two directions: transverse and longitudinal cross-sections. A longitudinal cross-section shows the progress of the pier. The pier corresponds to a raising from the minimum level of the berm to a meter above the water level and its crest width is 6 m.

The design process of the berm was performed as follows:

– Establish the minimum dimensions of the berm required with a safety factor of 1,15 for a failure in post-liquefaction condition resulting from seismic solicitation;
– Determine how the berm should be constructed;
– Do analyses to confirm if the berm could be built easily depending on the scenario or if stage construction is required.

3.4 *Design of the COW*

This section presents the general requirements for the COW and trenches required for the COW construction. As explained in section 3.3, the axis

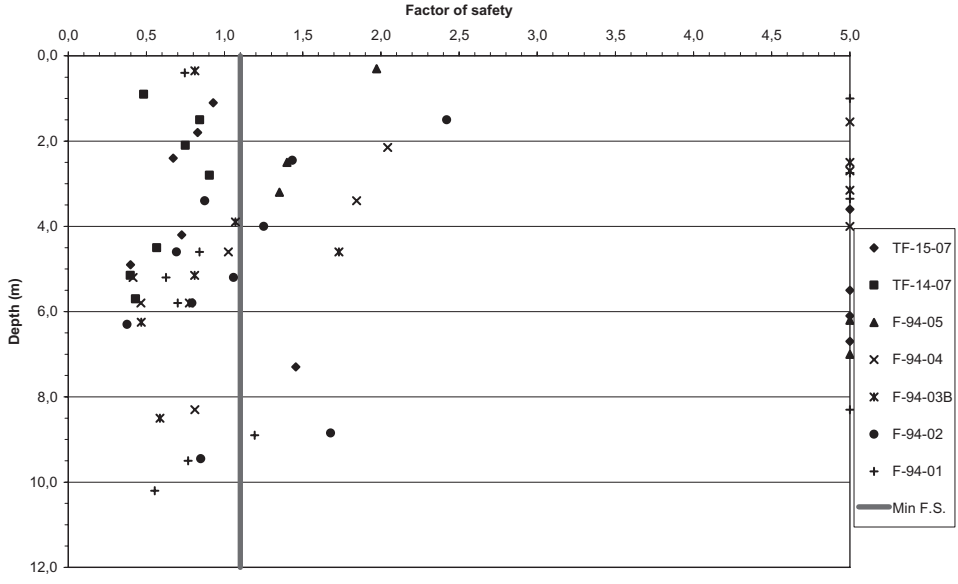

Figure 5. Factor of safety against liquefaction for crest boreholes.

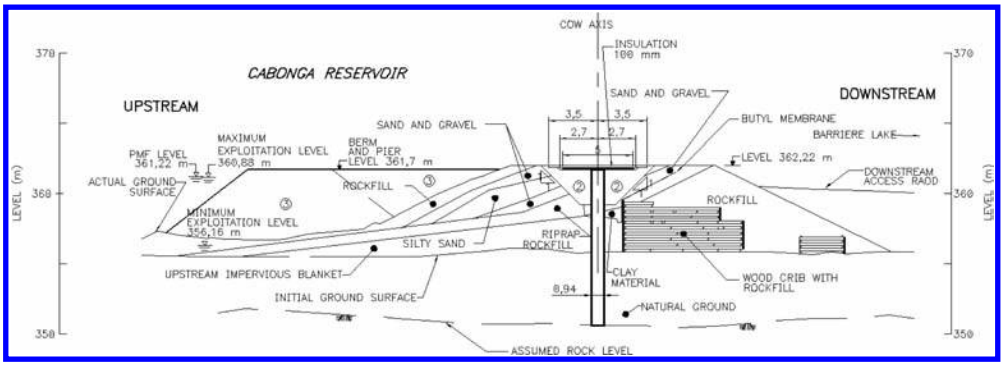

Figure 6. Typical cross-section after 2009 rehabilitation.

location was moved between technical pre-project study and engineering design. The new axis was 1,5 m upstream of 1994 borings (see figure 6). For these boreholes, the wood crib structure was not intercepted and it was considered safe to place the axis there.

The maximum gradient criterion for a cement-bentonite wall is 15 (ICOLD 1985 and 2002). At the Barriere Dam, the minimum COW thickness for this gradient is 300 mm, but for construction purpose the thickness is 900 mm. In fact, it is reasonable to expect at COW location, boulders of about 1 m according to the regional geological analysis performed, even though the 1994 boreholes data showed that few large boulders was intercepted. The thickness of 900 mm was selected to enable the excavation of the anticipated boulders.

The unconfined compressive strength of the cement-bentonite after 7 and 90 days of curing had to be at least 125 kPa and 300 kPa respectively. The plastic deformation after 90 days of curing, determined with Consolidated Isotropically Drained triaxial (CID) test under a lateral pressure of 100 kPa and a deformation rate of 0,1% per minute, had to be at least 6% without fissuring. The permeability coefficient had to be equal to or less than 1×10^{-6} cm/s measured after 90 days of curing.

The 2007 soils investigations showed that the right embankment had a lot of boulders near the abutment. Consequently, it was decided to embed the COW in till filled cut off trenches made at each abutment before the COW. Large boulders observed in investigations test trenches would be easier to removed in a large cut off trench than with COW. The fines content of the till is between 16 and 60% and maximum particle size is 200 mm. The lift thickness is 300 mm and compacted with 4 passes of a smooth drum vibratory roller (10 metric tons at drum). The filters are made of sand and gravel with maximum particle size of 80 mm and fines content less than 5%.

Gravitational soils injection (see 2.2.2 and 2.2.3) was planned to plug the probable voids between the stones of the 1966 riprap. But, with the new axis location of the COW, it was proposed by the owner to do a replacement trench to remove the 1966 riprap instead of injection work. The replacement trench was possible at this axis location because the excavation required is less than 3 m depth. It was considered less risky to do the trench compared to the contingencies related to injection work. The replacement trench was filled with sand and gravel. The fines content is less than 5% and maximum particle size is 300 mm. The lift thickness is 450 mm and compacted with 4 passes of the 10 tons roller.

4 CONSTRUCTION

The rehabilitation works of the right embankment of the Barriere Dam were done in two stages. The first was to build till filled cut off trenches at each abutment and replacement granular trench between the till filled cut off trenches. It was considered preferable to plan the work during the period when the reservoir water level is the lowest for the achievement of till filled cut off trenches, rather than requiring owner to adapt its reservoir management for the rehabilitation. This period was March, before the thaw begins. The second was to build an upstream berm and COW. In addition, the downstream access road of the dam was rehabilitated to meet the Ministry of Transport of Quebec (MTQ) standards.

4.1 *COW*

The preparatory works for the first stage began February 26, 2009 with the snow removal of the forest access road (±30 km) to the dam from the highway between Montreal and Val d'Or, Quebec. The excavation began on March 5, 2009 with the replacement trench towards the right abutment and the trench backfill was completed in March 27, 2009. The till filled cut off trenches were made between March 11 and 26, 2009.

The temporary facilities and preparatory works for the achievement of COW (second stage) were done between August 12 and 24, 2009. Destructive borings were drilled with a spacing of 6 m on the COW axis to confirm the bedrock level. The COW was made between August 26 and September 14, 2009 and the work plat-form was the dam crest.

The equipment required on site was indicated in the specifications because of previous experiences on others dam site and to be sure to get bedrock in case of large boulders. Excavation equipment that had to be on site included:

- Excavator to begin the excavation of a panel to a minimum of 3 m depth and enough capacity to excavate 900 mm width trench and remove boulders or pieces of boulders;
- Hydraulic clamshell to excavate soils and remove boulders or pieces of boulders;
- Chisel with a minimum mass of 12 metric tons to break large boulders.

Finally, the constructor did the trench excavation with only the long reach boom excavator (Koehring 1066 D) and the chisel.

Figure 7 shows the COW profile. The characteristics of COW are shown in table 3 and table 4 shows panel information. The final area for the COW is 1 349 m^2 and its theoretical width is 0,94 m (bucket width). The mean and

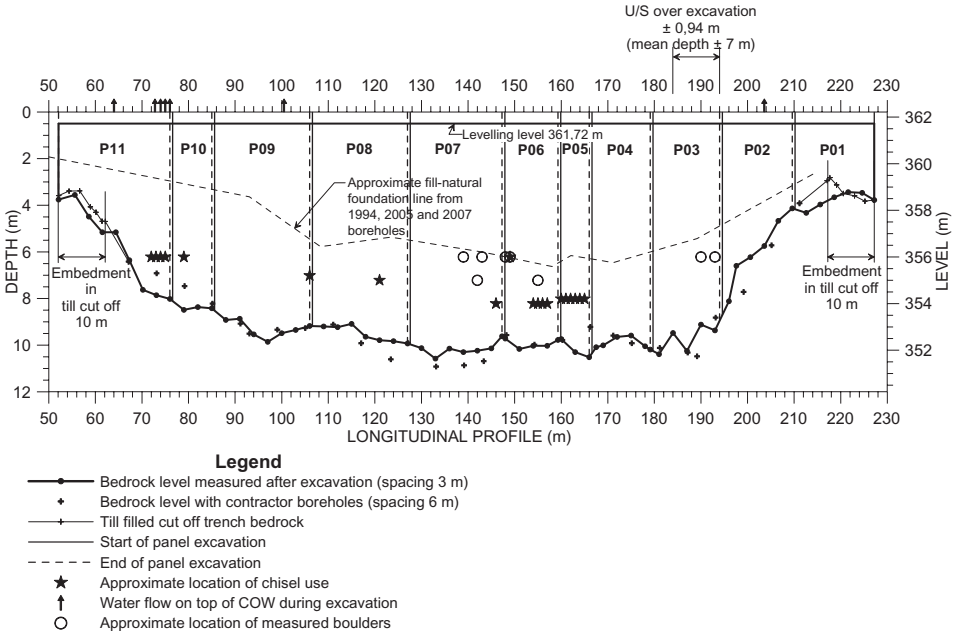

Figure 7. Cement-bentonite COW profile.

Table 3. Characteristics of Barriere Dam COW.

COW length (m)	175,1
Crest level (after leveling) (m)	361,72
Number of panel	11
Panel length (m)	7,3 to 24,5
Mean panel length (m)	16,5
Maximum depth (m)	
Dam crest to bedrock	10,6
COW crest to bedrock (after leveling)	10,1
Mean COW depth (m)	
Dam crest to bedrock	8,3
COW crest to bedrock (after leveling)	7,8
COW area (after levelling) (m^2)	1 349
Minimum COW width (mm)	940
Unconfined compressive strength (kPa)	
7 days of curing (mean)	131 (79 to 233)
91 days of curing (mean)	303 (232 to 405)
Hydraulic conductivity (with hydraulic gradient of 7,08) (cm/s)	$1{,}15 \times 10^{-6}$
CID test (deformation without fissuring) (%)	Min. 6

Table 4. Barriere Dam COW panel information.

N°	Date	Longitudinal profile (m) Start	Longitudinal profile (m) End	Area (m^2)
P01	08/26/2009	0+227,2	0+209,6	58,2
P02	08/27/2009	0+210,2	0+194,0	95,8
P03	08/27/2009 08/28/2009 08/31/2009	0+194,6	0+179,1	128,8
P04	09/01/2009	0+179,7	0+166,0	122,9
P05	09/02/2009	0+166,6	0+159,3	64,8
P06	09/03/2009	0+159,9	0+147,3	113,8
P07	09/08/2009	0+147,9	0+127,0	196,9
P08	09/09/2009	0+127,6	0+106,0	188,5
P09	09/10/2009	0+106,6	0+085,0	184,0
P10	09/11/2009	0+085,6	0+076,0	70,7
P11	09/14/2009	0+076,6	0+052,1	124,4

maximum depth are 8,29 and 10,6 m respectively. In figure 7, the locations where the chisel was used are indicated and the locations of measured boulders. In fact, 11 boulders of important size were measured. They were found in majority at the natural foundation and upstream impervious blanket contact. On all of these boulders, some organic materials were observed. The biggest measured boulder is 1,90 m × 1,20 m × 0,90 m, but for the others at least one dimension is about 1 m.

During excavation, some water flows were observed on the top of COW. These flows result probably from the compression of entrapped water, from a preferential flow zone, in the slurry

and the water goes up because its density (1 g/cm^3) is lighter than slurry density (min. of 1,19 g/cm^3). After slurry hardened and COW was levelled to its final level, no indication of these flows were located.

4.1.1 *Quality control*

The method used to confirm bedrock level was to introduce a H-beam in the slurry trench to measure the depth at each 3 m along the axis. This depth was compared to results of the destructive borings.

The bentonite slurry was controlled twice a day by density and Marsh viscosity tests. The cement-bentonite grout was tested at the exit of batch plant and in trench continuously because the laboratory technician was on site. The tests done were density, Marsh viscosity, bleeding (release of water from the grout), sand content for sample taken in the trench. Five samples of grout were taken for unconfined compressive strength, one permeability and CID test.

4.2 *Berm*

The pier and the berm were built between July 31 and August 20, 2009. The quarry is located about 400 m from the dam site. A total of 7 200 m^3 of random rockfill was placed. For the pier construction, the rockfill was partially selected for the upstream side to remove the smallest particles to control the suspended matter in water. The contractor used a turbidity curtain to control the suspended matter plume.

5 CONCLUSION

The rehabilitation of an existing structure is complex, as presented in this paper. The behaviour of a composite structure is difficult to understand. Several problems occurred during engineering progress. The search for as built and historic documents was arduous and yielded incomplete results but remains a *sine qua non* step in any serious study of old structures.

The realization of a cement-bentonite COW to stop infiltrations is a good but expensive solution. A few days after the completion of the COW, the visible infiltrations had stopped. However, for this dam, it is not necessary to control infiltrations because the waterlost goes to downstream power plants anyways and does not have significant economic value. It would have been possible to make a downstream berm with an inverted filter to ensure the stability of the structure. But, due to the remote location of the dam and its past history, the decision was made to build a positive cut off wall.

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Asphalted concrete core for rapid construction of SHUR River Dam (Design and construction)

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ABSTRACT: Kerman is a mid-east province of Iran with a lot of different mines. Copper mine is the most famous ones which largely have located at SARCHESHMEH area with 160 km. distance to city of Kerman. To provide water for copper production a lot of activities have been done from 30 years ago to now. The latest one is design and construction of a dam on SHUR River that is near the plant and has a pipeline for conveying water from the dam to the SARCHESHMEH plant area.

SHUR river dam is an Asphalted Concrete core Rock fill Dam (ACRD) that provides water storage for the SARCHESHMEH mine processing plant. The site location is at a distance 10 km. downstream of the plant with an 86 meter height and 34000000 m^3 capacities. The most important reason that impacts to decide selection of ACRD dam type have been: (1) rate of rapid construction for store water especially during construction, (2) low quantities of clay and (3) more earthquakes in this area.

1 STUDY AND DESIGN

Site geology of SHUR river dam is generally igneous origin and consists of inter-bedded ANDESITE and tuff. Water pressure tests of dam foundation have indicated that mostly the foundations are tight and would require limited grouting. However in right abutment with some high LUGEON values were found some fractured zones. Water tightness of whole foundation and abutments consist of three rows of grout holes, one as a curtain with min. depth 40 m and two shallow blanket grout holes with max. 15 m. depth. Figure 1 shows the general section of SHUR river dam.

The main quarry for production of embankment construction material (rock fill and filter) was a rock mass of igneous that consisting of ANDESITE basalt and has been located 1 km. south east of dam site.

For flood management two kinds of flood has been predicted (1) in time of construction phase with capacity of 1:10 yrs and has been adopted for design of a 1.5 meter pipe diameter as a diversion system (2) operational phase that has been concluded to a 110 meter length spillway channel

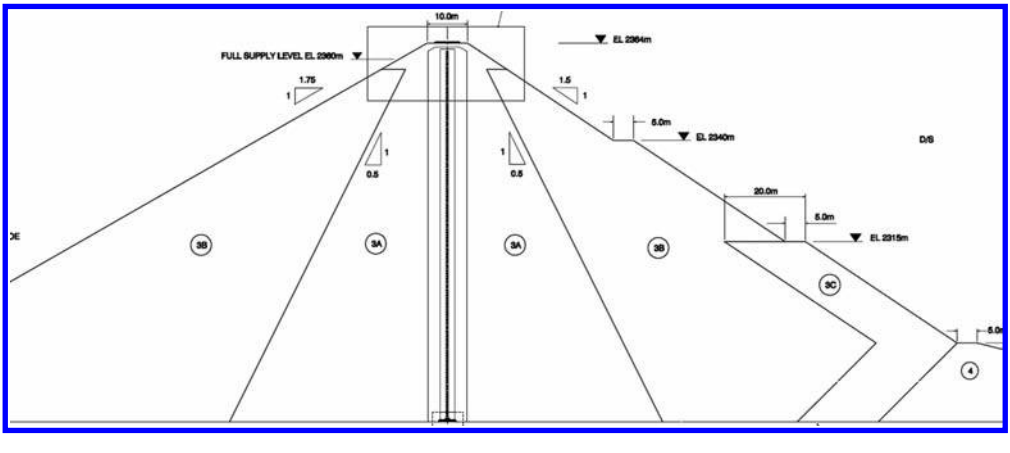

Figure 1. General section of SHUR river dam.

Table 1. Targeted min. F.S. in static analysis and results.

Description	Target F.S.	Upstream	Downstream
Full supply level 2360 m	1.5	2.38–2.42	2.05–2.08
Peak flood level 2364 m	1.3	2.44–2.49	2.02–2.04
Minimum operation level 2315 m	1.3	2.1	–
End of mine life 2360 m	1.3	–	2.13

Table 2. Deformations with different analysis.

Analysis method	Maximum deformation
Software QAKE/W & Slope/W	0.38 m
Seed simplified procedure	0.66 m
Finite difference by software FLAC	1.47 m vertical & 1 m horizontal

at elevation of 2360 and capacity of 240 meter cube per seconds.

Four conditions in static stability has been predicted and based on standards, targeted min. factors of safety before stability analysis be determined. The static stability analysis with slope stability software Slope/W and Limit Equilibrium Method was used.

The results has demonstrated that body of SHUR dam was conservatively stable for all operating conditions in the static state.

In seismic stability two earthquakes has been analyzed: (1) Operating Basic Earthquake (OBE) with average recurrence interval 1:500 yrs and (2) Max. Design Earthquake (MDE) with average interval 1:1000 yrs. In full supply level F.S. = 1 at OBE and F.S. smaller than 1 at MDE has been calculated and accepted.

Maximum deformations would be critical on upstream slope and be predicted from each of analysis method as a result of seismic induced loading from MDE.

The preceding analysis has demonstrated that embankment was stable and unacceptable deformations were not predicted as a result of seismic loading.

Management for stream flow during the construction and period of operation has consisted of the following elements:

- A 17 meter coffer dam with a clay core and a crest elevation equal to 2302 m.
- One 1500 mm diameter concrete encased steel diversion pipe.
- A 5 m wide diversion channel spillway with elevation level at 2300 m. to increase construction protection from 1:2 yrs to 1:10 yrs.
- A 110 meter main spillway with 2364 m crest elevation and capacity of 240 m^3/sec in time period of 1:1000 yrs.

With coffer dam and its spillway channel in time period of 1:10 yrs the peak flood height would be 26.6 meter and its flood capacity 84 m^3/sec so the risk of overtopping in this case would be zero.

For filling the dam during the construction and decommissioning of the diversion pipe, first the crest elevation of embankment must be greater than 2315, second the return water pumps (two pontoon pumps with discharge of 0.944 m^3/sec) have to commission and third, the best time is in summer so the risk of overtopping would be low.

Settlement monuments for measurement of lateral and vertical movement, Piezometers for monitoring of foundation Pore water pressure and ten interception pipes to detect seepage and leakage from core panels have been designed.

The body of dam which designed consisted of a 0.6 m. wide asphalted concrete core with maximum size of aggregates about 16 millimeter and bitumen content of 7.8%. Figure 2 shows the contact between asphalt core and concrete plinth and how the drainage pipes placed in downstream of core and upstream of a collection wall.

2 CONSTRUCTION (2008–2010)

Construction of SHUR river dam has started from January of 2008 and still is going to continue. End of 2010 shall be the time of completion and probably the filling of dam in end of 2010 will start.

2.1 *Diversion canal*

At time of dam construction the flow of river had to divert to make a secure area for construction. A concrete encased pipe with 1.5 meter diameter and 440 meter length at the river bed of SHUR River in January 2008 to June 2009 has been done.

First the alignment of pipe by blasting has been excavated and after some foundation preparation

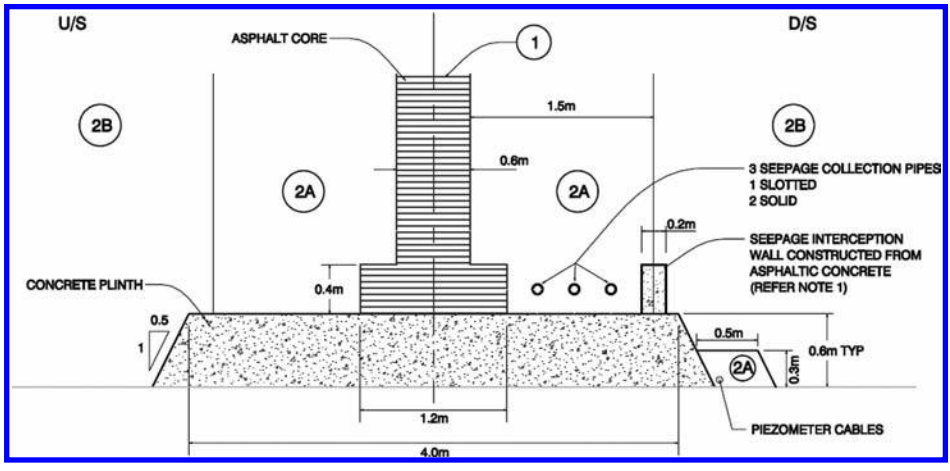

Figure 2. Prepared by ATC.

to reach an exposed rock, a suitable foundation for pouring foundation slab of diversion pipe has been provided. The diversion pipe was a thin steel pipe of a circular cross section with an internal diameter of 1.5 meter. The pipe was capable of supporting its own weight at rest and during pouring of concrete has not been occurred any cross section deformation.

The Sulfate Resistance cement has been used and so the Structural concrete had a minimum 42 days characteristic compressive strength of 40 mega Pascal with a maximum slump of 60 millimeters. There were 82 times of concrete part and so 82 samples had been taken. Figure 3 are showing the average of monthly results according to 82 samples. Figure 4 are showing the structural section of diversion canal. Some water stops from a non Toxic Hydrophilic rubber with ability to expand by 5 times its original volume when exposed to water just in centerline of SHUR river dam has been used.

The embankment has a series of seepage collection pipes to collect seepage from core of dam as a kind of monitoring. the pipes consist of DN90 PE80PN20 HDPE pipe and in total, six continuous lengths of pipe has been installed adjacent to diversion pipe for later connection to seepage collection pipes from the SHUR river dam embankment.

2.2 *Coffer dam*

For diverting the water from river to diversion pipe a coffer dam from July 2008 to June 2009 has been constructed. Height of coffer dam was 17 meter from river bed and its crest length about 175 meters. Its location was just upstream of dam slope and beginning of diversion canal entrance. Construction of the coffer dam embankment involved the placement of five zones of rock fill and earth. Zones 3A and 3B material consisted of rock fill and zones 2A and 2B have consisted from sands and gravels that have won from crasher as filters. Zone 1 consisted of clayey sands that have been won by free excavation from the borrow areas.

The most important part of coffer dam construction was clay placement. Maximum layer thickness of clay after compaction has not been greater than 150 millimeter and compaction density of layers at all the time was greater than 95% by HILF density ratio witch is a rapid test method. During clay placement of coffer dam, 12 plasticity index tests with average result of 18.3 have been done. The first layer of clay was at elevation of 2282 and the last one was at 2301 and following of that 98 compaction test during this activity has been done. Figure 5 shows the clay placement and its compaction.

The spillway of coffer dam has a length about 225 meter that started from 2 meters down of crest elevation of coffer dam and finished in downstream of main dam plinth. The exact role of this spillway was to control extra floods to avoid any flow to dam foundation during time of grouting. For excavation, blasting has been used.

2.3 *Concrete plinth*

The concrete plinth forms the foundation for asphalted concrete core of dam. The concrete plinth has been constructed on the prepared foundation and in a horizontal alignment with

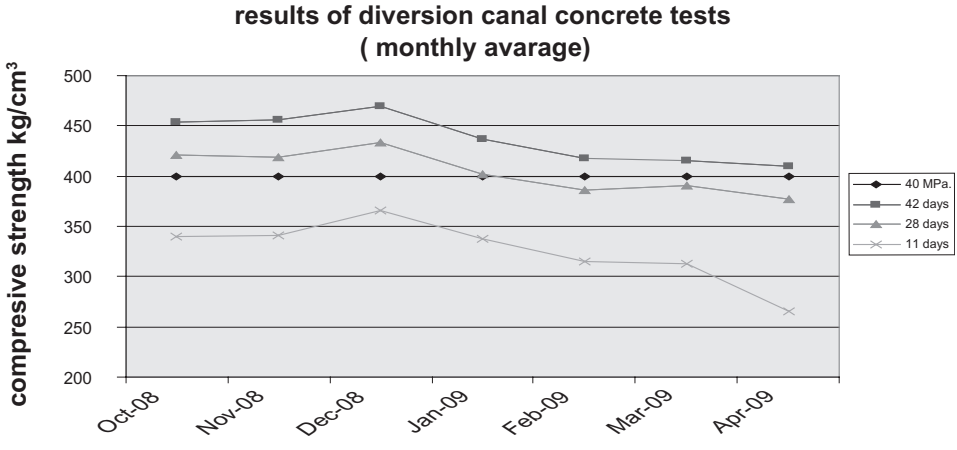

Figure 3.

Figure 4. Diversion canal during construction.

dam crest. It has been started from September 2008 to October 2009. The maximum thickness of plinth was 1.4 meter and minimum of that 0.6 meter. It has been started from the lowest elevation and advanced equally along each abutment. Some anchor dowels (T24 bars) have been installed within the plinth with a minimum depth of 3 meter to ensure stick concrete and foundation to each other. Compressive strength of 42 days concrete had been more than 40 mega Pascal. Figure 6 shows the average monthly results among of 83 samples which have been taken during concrete placement. For making water tightness between the joints, two water stops have been constructed at each construction joint in concrete plinth. These water stops have composed with a 100 mm diameter cored hole that filled with bituminous mastic. Then at the upstream of each construction joint with a bitumen geotextile tape have been covered and a bituminous mastic layer with 5 to 10 millimeter thickness over the plinth surface applied. The mastic consisted of bitumen (20%), filler (15%), sand (65%) and STEARIC Acid (2% of bitumen weight).

2.4 *Asphalted concrete*

Prior to commencement of work on the embankment, a trial which demonstrated the asphalted concrete mix design and construction methods in site has been done. First a 25 meter concrete slab at 4 meter wide constructed and after applying a 2.2 meter width of mastic, 4 layers of asphalt over on that has been done. After completion of placement, 100 millimeter cores at 11 locations drilled

Figure 5. The clay placement at coffer dam of SHUR river dam.

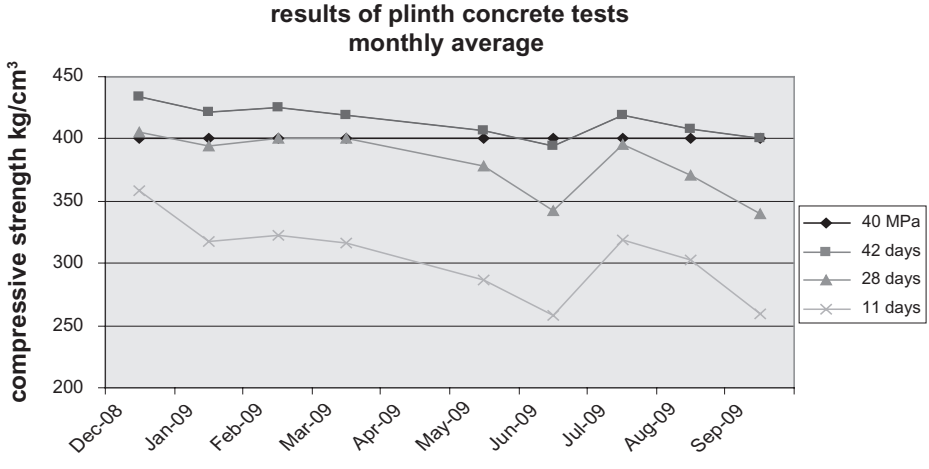

Figure 6.

and each core for inspection and testing were logged. The most important test on the samples was void ratio that in all of the samples had been less than 3% which were accepted.

For asphalt core lying, the concrete plinth must be cleaned and by some heaters asphalt mastic be applied. Asphalt is transporting by pay loader and two transition zone called 2A and 2B would place around core. A paver's machine that equipped with camera and in front the operator to steer the machine along the center line of core. The asphalted concrete is placing with a machine at uniform thickness which after compaction the layers have been 20 centimeters. All the time and in abutment for first layer the formwork has been used. Figure 8 shows the paver machine that has been used in SHUR river dam.

2.5 *Results of tests*

Placement of asphalted concrete core according to mix design started at 18/10/2009 and continues up to now. Table 3 shows how the placement of core layers in main dam and saddle dam has been done. The number of layers per day, the bitumen percent that has been find by tests in asphalt plant and after placement and void ratio by marshal test, laboratory and by coring are the most important factors which has been calculated and compared to each others.

It can be seen from table 3 that first it is possible to place 4 layers of asphalted concrete core in one day that compare with clay placemen, the asphalt placement per day is twice of clay. Secondly the test for bitumen analyze in both method

Figure 7. A Trial test of asphalt concrete in site of SHUR river dam.

Figure 8. PAVER machine at SHUR river dam.

(sample mix from plant and test in two different laboratory by method of combustion and by centrifuge) all the time shows bitumen percent around mix design (7. 8%) and voids by three method (two methods by marshal test in two different laboratory and the third ones by coring) are under 3% (mix design specification value) and mostly not more than 2 percent which it can be a good result.

Asphalted concrete core could be placed by 4 layers per day, so it is a kind of revolution of water tightness construction in earth and rock fill dams.

Figures 9 and 10 show the result of tests about different methods for bitumen content and void ratio.

4 CONCLUSIONS

- The most popular fast method for water barriers in earth and rock fill dam is asphalted concrete core.
- The asphalted concrete core dams are very helpful in some climates like the SARCHESHMEH area with 2400 meters elevation from sea level which is very cold and has long winters.
- Productions of some material are under the control of the site and some of them are not. The contractor can modify the aggregates production in crasher and making the asphalt in asphalt plant but the bitumen is coming from

Table 3. Results of 13 days asphalted concrete core placements.

No.	Date	Layers	Bit.%	Bit.%	Void%	Void%	Void%
1	18/10/2009	1	7.7	–	1.01	–	1.87
2	19/10/2009	1	7.98	–	1.54	–	2.09
3	22/10/2009	2	7.51	7.5	0.85	1.15	1.84
4	25/10/2009	4	7.66	7.9	0.61	0.87	–
5	26/10/2009	2	7.26	–	0.65	–	–
6	27/10/2009	3	7.29	–	1.15	–	–
7	28/10/2009	4	7.44	7.67	0.69	1.01	–
8	29/10/2009	4	7.2	7.82	0.91	1.07	–
9	30/10/2009	3	8.14	8.4	1.24	1.07	1.02
10	1/11/2009	4	7.56	7.85	0.97	0.85	–
11	2/11/2009	2	8.04	8	1.12	0.9	1.65
12	5/11/2009	3	7.4	–	0.78	–	–
13	6/11/2009	4	7.95	–	0.97	–	–

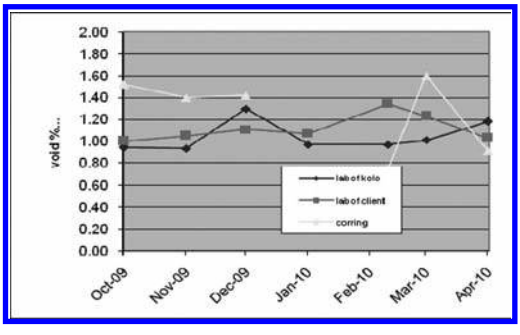

Figure 9.

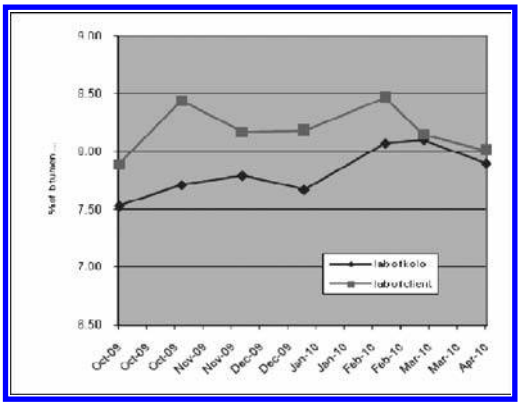

Figure 10.

some refineries out of the site, so the special notice have to need and regular test must to apply that.

– The most important view point of ACRD is mix design. The experiences show at least 2 to 3 months of time need to reach the best mix design. In the beginning of construction time period this is too much.
– Training for asphalt plant technician, pavers and roller driver have to do in time of doing the mix design.
– There is no time for training an asphalt laboratory technician. So immediately in time of finding the mix design one trained laboratory technician which had do some mix design before must start in laboratory of site.
– Quality controls for void ratio of asphalt are the number one of supervision team notice and have to be carefully to this point of view.
– Pavers machine are very sensitive to repair and maintenance, so special program for maintenance of this machine have to be done because the progress of job depends to only one PAVER that is in the site.

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Recuperación de niveles de seguridad de una presa antigua

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ABSTRACT: Los niveles de seguridad de presas antiguas pueden verse reducidos por diversas circunstancias, deterioro de los hormigones por reacciones química o agentes meteorológicos, por alteración de las características del cimiento, etc. Los autores presentan, sobre la base de ejemplos concretos, una sistemática de actuación que arranca de la detección y análisis de indicios, pasando por el análisis de seguridad de la presa y culminando con las actuaciones correctoras.

1 LAS PRESAS ANTIGUAS

La construcción de presas se rige en la actualidad por criterios muy estrictos, tanto en su diseño, como en la obra y posteriormente durante su explotación. Lamentablemente esto no siempre ha sido así. Aún reconociendo, como es justo, el mérito y buen hacer de nuestros predecesores profesionales, los medios de que se disponía y el nivel de conocimientos de tiempos pretéritos quedan lejos de los actuales.

Así, las presas antiguas, o sea las de más de 50 años de vida, adolecen, en general, de ciertas deficiencias que podemos concretar en: a) Deficiencias de diseño; b) Deficiencias constructivas; c) Documentación escasa o nula y d) Instrumentación y auscultación escasa o nula.

La presente ponencia muestra, sobre una experiencia concreta, el propio proceso degenerativo y su detección, su evaluación desde el punto de vista de la seguridad de la presa y las actuaciones encaminadas a la recuperación de los niveles de seguridad exigibles. El caso tiene el interés adicional de combinar dos fenómenos diferentes: el ataque químico por sulfatos y la apertura de las juntas de hormigonado por deficiente ejecución.

La presa que nos servirá de referencia está situada en el río Rivera de Huelva, afluente por la derecha del Guadalquivir. Está asignada al abastecimiento de Sevilla y su área metropolitana, siendo su titular la Empresa Metropolitana de Abastecimiento y Saneamiento de Aguas de Sevilla (EMASESA). Se trata de una presa de hormigón, con perfil de gravedad y casi 60 m de altura sobre cimientos. Fue definitivamente puesta en servicio, con su aspecto actual, en 1956.

Figura 1. Presa de La Minilla (Sevilla, España).

2 PROCESO DEGENERATIVO. CARACTERIZACIÓN

En lo que sigue, nos vamos a referir a tres fenómenos degenerativos, habituales en una u otra forma en las presas antiguas y que pueden poner en peligro su seguridad a medio o largo plazo.

Uno de los problemas más frecuentes que se presentan en las presas es la degradación de los

hormigones por ataque químico, ya sea interno, por reacción de los áridos con la pasta de cemento, ya sea externo, por la presencia de aguas sulfatadas principalmente. Nos vamos a centrar en el caso de ataques por sulfatos, más en concreto en los sulfatos derivados de la oxidación de piritas y otros sulfuros metálicos.

Las reacciones químicas que nos ocupan son bastante dañinas en general, pues generan productos notable o fuertemente expansivos que fracturan el cuerpo de presa. Sin embargo, hay que hacer alguna precisión al respecto. Las diversas normas establecen unos porcentajes máximos, a partir de los cuales la concentración de sulfatos es perjudicial para el hormigón. Esto supone que el sulfato se encuentra uniformemente distribuido por la masa del hormigón, lo cual es imposible, salvo que se encuentre disuelto en el agua de amasado.

La limitación porcentual anterior no tiene en cuenta la distribución del sulfato en el cuerpo de presa. En la selección de áridos para la fabricación del hormigón se tenía en cuenta la apariencia de los mismos, su dureza, etc. y así se desechaban los materiales yesíferos, pero no ocurría lo mismo con los que contenían piritas, calcopiritas, pirrotinas, etc. que son igualmente peligrosos. Ello se debía a que frecuentemente van asociados a cuarcitas, pórfidos, etc. válidos para la fabricación de hormigón. Cualquiera de los minerales citados se presenta en forma de filones, por lo que su presencia en el hormigón puede ser muy concentrada en determinadas zonas y nula en otras. Ello podría dar concentraciones medias por debajo de los límites establecidos en las normas y a pesar de ello provocar daños importantes.

La pirita es un bisulfuro de hierro que adopta la forma de cristales cúbicos, que se han formado en pequeñas fisuras de la corteza terrestre. La oxidación de las piritas produce sulfato férrico y ácido sulfúrico. Ambos actúan sobre el aluminato tricálcico, A3C, dando lugar a una nueva forma cristalina, la ettringita, que es un sulfato hidratado de calcio y aluminio, cuyo principal problema es que, en su formación, expande hasta un 227%. En este momento pueden ocurrir dos cosas. Si la pirita se encuentra muy repartida por toda la masa y en cantidad importante, el conjunto del hormigón mostrará notables deformaciones externas, fisuración superficial, etc. Si, por el contrario, la masa de pirita es reducida en comparación con la masa de hormigón y concentrada en algunos puntos, la degradación no se manifestará al exterior, salvo en puntos aislados, próximos a la superficie.

En efecto, un pequeño cristal de pirita de 1 gr reaccionaría con el A3C de unos 2 cm^3 cúbicos de hormigón. Las tensiones que se producirían serían muy altas. El orden de magnitud sería de varios centenares de bar (en esta evaluación influye mucho la dosificación de cemento, la granulometría del árido, la compacidad y los parámetros elásticos consiguientes: módulo de elasticidad, coeficiente de Poisson, etc.). Se ha estimado que, al menos, un volumen 5 cm^3 quedaría prácticamente pulverizado y otro volumen mucho mayor disgregado. Por tanto, la degradación del hormigón se concreta en la formación de oquedades que ocasionalmente pueden fusionarse, si la distribución y cantidad de pirita lo propician. En el caso de La Minilla, la presa se encuentra situada en la faja pirítica de Sierra Morena. La presencia de pirita y otros sulfuros metálicos está garantizada, aunque la aparición de cristales de gran tamaño es poco frecuente. Si determinamos la concentración media de pirita por volumen de hormigón, resulta ser francamente baja, sin embargo ocasionalmente se encuentran zonas localizadas con notable concentración de pirita que desencadena los procesos degenerativos del hormigón con gran intensidad.

El segundo problema que podemos encontrar es el derivado de las técnicas constructivas al uso en la época de construcción y que impedían la correcta unión entre tongadas. Además los cementos de principios del siglo XX tenían elevadas temperaturas de fraguado que incidían negativamente en la calidad de dichas uniones. Estas juntas son "rebanadas" más o menos uniformes en espesor y constitución que: a) Son vías preferenciales de filtración; b) con el paso del tiempo, una filtración no controlada termina por lavar la junta; c) en todo caso constituye una forma de aumento de la subpresión en el propio cuerpo de presa; d) aunque la junta debe quedar interceptada por planos de drenes intercalados entre las diversas galerías de visita, es habitual que dichos drenes queden obstruidos por la cal y e) por todo lo anterior estas juntas de hormigonado (retomas) pueden llevar a la presa una situación no deseable, incluso peligrosa, poniendo en juego su estabilidad a deslizamiento, pudiendo combinar mecanismos desestabilizadores a vuelco.

El tercer proceso degenerativo de gran importancia es la pérdida de eficacia del sistema de drenaje y control de subpresiones. Los diversos estudios realizados por los autores muestran que dicha eficacia sigue una ley de tipo exponencial decreciente de forma que, como media, cada quince años debería hacerse una campaña de regeneración de los drenes, ya que en dicho intervalo de tiempo los drenes pueden haber disminuido su capacidad en un 60%.

3 DETECCIÓN DE DAÑOS, EVALUACIÓN DE LA SEGURIDAD Y MEDIDAS CORRECTORAS

Los procesos expansivos derivados de reacciones entre los áridos, el cemento y el agua suelen manifestarse y detectarse fácilmente por la magnitud del daño y la abundancia del árido reactivo, en especial las reacciones tipo álcali-árido. No ocurre siempre lo mismo en el caso de ataques por sulfatos. En efecto, la pirita es un mineral que aparece en las canteras en forma de filones de espesor muy variable. Esto hace que la proporción de pirita en el conjunto de los áridos pueda ser baja aunque concentrada en zonas del cuerpo de presa ("nidos de pirita") de distribución aleatoria. Es una diferencia fundamental en relación con la degradación por reacción álcali-árido.

Así, el fenómeno degenerativo puede desarrollarse de forma soterrada y ser de difícil detección con los medios habituales de auscultación. Hemos de recurrir a una observación muy cuidadosa, "detección por indicios", que posteriormente se complementará con una campaña de investigación exhaustiva que nos permita proyectar las medidas correctoras necesarias.

Otro problema en cuestión es el lixiviado y apertura de juntas frías. Los drenes del cuerpo de presa, además de luchar contra la subpresión del propio macizo de hormigón, constituyen un valioso medio de detección y evaluación de estas juntas. Es necesario realizar un seguimiento exhaustivo del comportamiento de estos drenes. La evaluación de los caudales drenados, la inspección periódica con cámaras de TV, la detección y evolución de costras de cal, etc., son reflejo del comportamiento de las juntas frías. Similares circunstancias se dan en el caso de los drenes de cimentación. En ambos casos, es fundamental el seguimiento del comportamiento de dichos drenes. En ambos casos es preciso medir los caudales y presiones con frecuencia, al menos una vez al mes. La reducción de caudales de filtración, lejos de tranquilizarnos, debe hacernos sospechar que el dren en cuestión se está obstruyendo. Por este motivo es recomendable inspeccionarlos con cámara de TV una vez cada cinco años, como mínimo.

Consideramos que estos tres casos compendian en gran parte los problemas con los que podemos enfrentarnos. La campaña de investigación, evaluación y corrección que a continuación se presenta puede servir muy bien de modelo para otras campañas La campaña constará de fases: Detección de indicios, detección y análisis de síntomas, reconocimiento sistemático, evaluación de seguridad y diseño de medidas correctoras.

3.1 *Detección por indicios*

La primera fase, "detección por indicios", es un proceso de sistematización de las observaciones, fundamentalmente visuales, que se pueden hacer en la presa y en su entorno. Su objeto es poner de manifiesto fenómenos degenerativos en el cuerpo de la presa o en alguno de sus órganos que pueden escapar a la auscultación instrumentada.

Los indicios por sí solos no implican la existencia de piritas en el cuerpo de la presa, pero si se observan varios de ellos simultáneamente, es motivo suficiente para continuar e intensificar la inspección.

Indicio 1: En los puntos donde aparezcan filtraciones o donde se hayan producido éstas aparecen depósitos ferruginosos.

Indicio 2: En un entorno de varios kilómetros aparecen filones con pirita.

Indicio 3: En las canteras que aportaron el árido para los hormigones de la presa de la presa se encuentran restos de pirita.

Indicio 4: En las galerías de inspección hay zonas con olor a sulfhídrico (salvo problemas de ventilación no será muy intenso)

Indicio 5: Al tratar el agua procedente de filtraciones con un indicador de pH, éste da muestras de acidez. Esto sería señal de un proceso de sulfatación en marcha. Los puntos a controlar serán principalmente aquellos en los que se venga observando un caudal creciente de filtración y aquellos que se encuentren en zonas en las que haya olor a sulfhídrico.

Por muy bueno que haya sido el proceso de hormigonado, es fácil localizar, como poco, unas pequeñas fisuras semihorizontales que se corresponden con las llamadas "juntas frías" o "retomas". Lo que no es fácil es evaluar el estado de deterioro de esas juntas. La evolución de los caudales recogidos por los drenes del cuerpo de presa nos dan una buena estimación del grado de lixiviado/apertura de la junta fría. Una variación brusca en el caudal drenado significa una apertura de la junta. Cuando las filtraciones aparecen en el paramento de aguas abajo, el fenómeno se encuentra ya bastante avanzado, a veces favorecido por la obstrucción de los drenes del cuerpo de presa. Es por tanto muy importante controlar dichos caudales de forma exhaustiva.

En el caso de los drenes de cimentación, el fenómeno es similar al anterior aunque más difícil de conceptualizar por la irregularidad del terreno, tanto física como química. En cualquier caso, se pone de manifiesto que los drenes como medio

de auscultación, por un lado, y como sistema de seguridad, tienen tal importancia que han de tener un seguimiento muy detallado, no solo en las mediciones, sino también de su estado de conservación y eficacia. En este sentido es muy recomendable la inspección periódica con cámara de televisión. Lo mismo cabe decir del sistema de piezómetros que lleva asociado.

3.2 *Análisis de indicios*

Una vez comprobado que hay varios indicios concurrentes, procede su estudio y el diseño de una campaña de investigación.

En el caso de la presa de La Minilla concurrían los siguientes indicios: 1) Manchas ferruginosas frecuentes en las paredes de las galerías; 2) La presa se sitúa en la faja pirítica de Sierra Morena; 3) Hay canteras abandonadas en el entorno de la presa en las que aparecen vetas de pirita; 4) En algunos tramos de las galerías, especialmente de la 4ª, hay un claro olor a sulfhídrico; 5) En los testigos extraídos de la cimentación aparecieron importantes vetas de pirita; 6) En dos afloramientos de agua en la galería 4ª, el agua mostraba cierta acidez; y 7) Los drenes del cuerpo de presa presentaba grandes costras de carbonato y en el paramento de aguas abajo aparecían afloramientos horizontales de agua.

Para realizar el análisis de indicios hemos de tener en cuenta unos factores positivos, o sea que favorecen el deterioro, unos factores negativos, esto es que lo frenan, y unos factores que lo ocultan.

Entre los factores positivos podemos señalar la porosidad y fisuración del hormigón, las frecuentes alternancias sequedad-humedad que se dan en el paramento de aguas arriba, las galerías y demás huecos internos de la presa; los flujos internos de agua, tanto más cuanto mayor sea la velocidad; etc. Los factores negativos son los contrarios de los indicados. Entre los factores que ocultan el fenómeno podemos indicar el carácter masivo de la presa que puede provocar una "implosión" del hormigón que pulverice éste y genere una oquedad, pero no se vea en el exterior y la fácil solubilidad de la ettringita, entre otros.

Figura 2. Testigo de sondeo hecho desde coronación. A 43,40 m de profundidad apareció el hormigón disgregado, sin restos de cemento y en una longitud de unos 9 m.

En el caso de La Minilla el análisis de indicios se basó en: a) Comparar el volumen de presa con el de las canteras que resultó ser menor, por lo que se concluyó que se había utilizado material procedente de la excavación; b) Comparar la concentración de pirita en cimentación y en canteras con la encontrada en el cuerpo de presa y con la que, por razón de estequiometría, correspondería a los volúmenes dañados. Así se concluyó que la pirita procedía efectivamente de esas canteras y de la cimentación; c) Comparar la frecuencia de la aparición de daños con la densidad de pirita encontrada, aceptando que el árido procedente de la cimentación, el más pirítico, se había colocado en la parte baja de la presa. La correlación resultó bastante alta y d) Comprobar el correcto funcionamiento de las compuertas, ya que los hinchamientos provocan frecuentes fallos en los mecanismos.

La última comprobación resultó fallida. Sin embargo al repasar la historia de la presa se pudo ver que ésta se ejecutó en dos fases, colocando los mecanismos en la segunda fase, lo que pudo salvar los mecanismos por haberse desarrollado el fenómeno expansivo en los primeros años de vida de la presa. Por otro lado, el material empleado en coronación era mucho menos rico en pirita.

Adicionalmente se realizó un estudio de la aparición de afloramientos de agua en el paramento de aguas abajo, comprobando que se correspondían sensiblemente con las zonas en que los drenes del cuerpo de presa estaban más obstruidos y concluyendo que las juntas frías inicialmente interceptadas por estos drenes ahora inutilizados estaba activas.

3.3 *Reconocimiento sistemático*

Tras los estudios previos y si éstos han aportado la certeza de un ataque sulfático, de lixiviado de juntas frías o similar, procede realizar un estudio, lo más detallado posible, para caracterizar el daño.

Los métodos geofísicos tienen importantes limitaciones y, para que resulten eficaces en estas circunstancias, tendrían que ser muy numerosos. La ejecución de sondeos de reconocimiento con extracción de testigos y posterior inspección con TV se manifiesta como la técnica más efectiva, tanto más cuando su ejecución se requiere para una posible inyección de regeneración del hormigón.

La técnica recomendada y ensayada por los autores es la siguiente:

a. Perforación de sondeos verticales, uno por bloque, desde la coronación de la presa y a poca distancia del paramento. En Minilla, se ejecutaron a un metro y medio aproximadamente de dicho paramento. Debe hacerse en diámetro 100 mm o superior y penetrar varios metros en el cimiento (se recomienda perforar el terreno al menos hasta un tercio de la altura de la presa). La perforación se hará a rotación con corona de diamante y extracción de testigo. Esto nos permitirá detectar grietas de tracción en el contacto terreno-presa, estado de juntas frías y estado general del hormigón, todo ello sin arriesgarnos a que entre agua en las galerías.
b. Visualización de los sondeos anteriores con cámara de TV y equipo de grabación.
c. En los casos de circulación importante de agua, se emplea un equipo de medida de flujo (caudales y distribución de puntos de entrada)
d. Cartografiado de anomalías. (surgencias de agua, actuales o antiguas, manchas ferrosas, abombamientos injustificados o zonas de fisuración en forma de piel de cocodrilo, etc.)
e. Con el mapa de anomalías, se diseña una red de investigación formada por taladros de 46 mm de diámetro y longitudes e inclinaciones variables de forma que se cubra el mayor volumen posible, sin que queden espacios superiores a los 2,0 m sin estudiar. La perforación debe hacerse a rotación con corona de diamante y extracción de testigo.
f. Limpieza con agua y e inspección con TV de todos los taladros
g. Revisión de la malla de perforaciones perforación de nuevos taladros en alguna zona para una mayor definición del problema. No debemos perder de vista que la ventaja del método que venimos exponiendo es que todos los trabajos que se vienen realizando serán aprovechables para la propia ejecución de medidas correctoras.

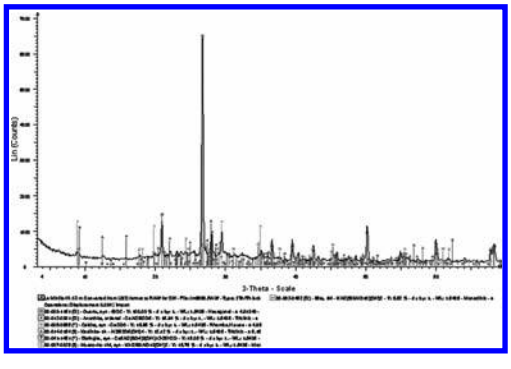

Figura 3. Difractograma obtenido sobre muestra extraída del testigo anterior. Muestra presencia de ettringita.

Es interesante señalar que resultó frecuente que, al perforar desde las galerías aparecieran importantes caudales de agua. Lo que confirmó la tesis de que la presa se encontraba sometida a una subpresión mayor de la debida, en su cuerpo de presa, por lo que era conveniente reducir la misma lo antes posible.

3.4 *Evaluación preliminar de la seguridad*

Después de revisar toda la información, se lleva a cabo en primer lugar una evaluación preliminar de los parámetros resistentes que gobernarían el deslizamiento en el contacto presa-cimentación, así como un análisis de sensibilidad en torno a los valores que se consideran la "mejor estimación".

Esta asignación de parámetros (aproximadamente 40 grados de fricción y 1 Kp/cm^2 de cohesión) se ha realizado a partir de referencias de literatura técnica. En particular de Link (1969), Fell et al (2005), Ruggieri (2004), Soriano y Sánchez (1998) y SPANCOLD (2002), además de la propia experiencia de los autores tras analizar el aspecto de algunos testigos tomados en la zona de contacto presa cimentación.

Con todo ello, los coeficientes de seguridad para embalse normal (cota 157,609) podrían oscilar, para los términos de fricción y cohesión, entre 1.0 y 3.4 en el caso de subpresión completa, y 1.45 y 4.90 en el caso de un drenaje eficaz (reduciendo las subpresiones en torno a un tercio de las teóricas).

Teniendo en cuenta que la Guía Técnica Número 2 del Comité Nacional Español de Grandes Presas recomienda coeficientes de 1.5 para la fricción y 5 para la cohesión en situación normal, cabe presumir lo siguiente:

- Con una probabilidad no despreciable, podría no cumplirse dicho estándar en este momento, siendo razonable suponer la presa tendría unos coeficientes más propios de lo que correspondería a una situación Accidental (1.2 y 4) que a una situación Normal (1.5 y 5).
- Resulta también bastante probable que, exclusivamente con un funcionamiento correcto desde el punto de vista de la impermeabilización y el drenaje (es decir, con un control eficiente de las subpresiones), la presa recuperase márgenes de seguridad próximos a los recomendados en la Guía Técnica N° 2.

Consecuentemente, cabe concluir de un análisis preliminar de la seguridad, pero suficiente para enfocar la toma de decisiones a la luz de la información disponible, que el conjunto impermeabilización-drenaje, el primero relacionado con evitar una degradación de las piritas y el segundo por su especial repercusión en la estabilidad general de la presa, debe ser el principal objeto de los

siguientes trabajos dado que permite "recuperar" un amplio margen de seguridad a deslizamiento y, a su vez, mejorar la durabilidad de la roca de cimentación y del propio hormigón del cuerpo de presa.

En cualquier caso, una evaluación rigurosa de la seguridad de la presa deberá combinarse con ensayos que permitan una mejor estimación de los parámetros resistentes.

3.5 *Diseño y ejecución de medidas correctoras*

Como se muestra en el epígrafe anterior, la mayor contribución a la estabilidad de la presa viene del correcto funcionamiento del sistema de control de la subpresión, esto es de la disponibilidad un buen sistema de drenaje, tanto de cimentación como "de piel" y un sistema de medida de subpresiones que certifique la eficacia del anterior. Sigue, en orden de prioridad, el sellado de las juntas de hormigonado que se encuentren abiertas y la inyección de las oquedades o zonas de menor compacidad.

Pese a esta importancia, el sistema de drenaje de cimentación de una presa no tiene unas reglas fijas: ni se dan diámetros mínimos o máximos, ni se concreta la separación y distribución en planta, ni se concreta nada sobre su longitud. Podríamos decir que "cada maestrillo tiene su librillo". Algo parecido ocurre con los drenes de piel.

En este epígrafe, exponemos: 1) la metodología seguida para la restauración, incluyendo su posible nuevo diseño, de las redes de drenaje (de cimentación y de piel) y del sistema de medida de subpresiones y 2) la metodología de inyección de oquedades y grietas (juntas de hormigonado o retomas) experimentadas por los autores.

Durante los trabajos de rehabilitación de sistemas de drenaje es habitual que nos encontremos algunos drenes que aporten un gran caudal de agua junto con otros cuya aportación sea escasa o incluso nula. La duda es evidente: ¿Rehabilitamos todos? ¿Aumentamos el número de drenes?

Aun disponiendo de amplia información sobre la geología de la cimentación es muy normal que la caracterización de las vías de agua sea escasa. La mera inspección con cámara de TV o la extracción de testigos no resulta suficiente. Tanto en el caso de La Minilla, como en el de la presa de La Ribeira de ENDESA, se han ensayado técnicas geofísicas que han permitido rediseñar satisfactoriamente la distribución y longitud de los drenes de cimentación. Para ello se han utilizado diversos tipos de sondas y los habituales data Logger y ordenadores portátiles adecuados a trabajos de campo en condiciones duras de trabajo. No se puede decir que una de dichas sondas, por sí sola, resuelva el problema, ya que según las circunstancias una aporta mejor información que las otras, pero muchas veces se complementan.

Las sondas ensayadas han sido las siguientes: Sondas de temperatura (los cambios de temperatura implican capas de aporte de agua); sonda de conductividad/resistividad (permiten correlacionar la concentración en sales con la litología del terreno); sonda de calibre (para medir la sección eficaz del dren), sondas radiactivas (para medida de radiación ganma y muy adecuadas para correlación litológica); sondas-caudalímetro de molinillo o de pulso térmico, según el orden de magnitud de los caudales, y sondas acústicas (proporcionan información sobre la orientación de las discontinuidades, la trayectoria del dren y el diámetro del propio taladro). La información proporcionada por estas sondas junto con la ya tradicional inspección por TV (hay también sondas ópticas de última generación que proporcionan una información adicional sobre la orientación del sondeo) permiten dimensionar y distribuir adecuadamente los drenes. En el caso de La Minilla, la utilización de estos medios permitió reducir el número de drenes inicialmente previstos a la mitad, aumentando sin embargo la eficacia del sistema de drenaje

Si importante es definir y mantener un correcto sistema de drenaje, no menos necesario es asegurar la compacidad y continuidad del cuerpo de presa. Las oquedades producidas en los ataques sulfáticos de que venimos hablando o la apertura de las juntas frías han de anularse lo antes posible.

En el caso que nos ocupa y en la primera fase de inyección se han utilizado resinas epoxídicas de muy alta viscosidad, entre 15.000 y 100.000 cp. La ventaja de estas resinas es que su viscosidad dificulta el arrastre por el agua, permitiendo realizar las inyecciones con embalse lleno. En la presa de La Minilla se hicieron varias pruebas

Figura 4. Un correcto diseño del sistema de drenaje aumenta notablemente la seguridad de la presa.

Figura 5. Distribución de taladros de inyectores provistos de obturadores.

comparativas entre microcementos, lechadas de cemento y resinas. Si bien el precio unitario de los primeros era sensiblemente más bajo que el de las resinas, el mucho mayor control de las segundas, junto con la resistencia a ser arrastradas, permitieron mantener los costes y asegurar la eficacia de la inyección.

Hay que señalar, sin embargo, que el empleo de resinas de tan elevada viscosidad, hace necesario el empleo de grupos de muy alta presión.

4 CONCLUSIONES

A la vista de las experiencias acumuladas y descritas en el artículo, podemos concluir que:

1. la aplicación de procedimientos de control estandarizados puede no ser suficiente;
2. es necesario contar con un equipo de profesionales experimentados para hacer las evaluaciones de seguridad de una presa;
3. la recreación del proceso constructivo es de suma importancia para alcanzar buenos resultados;
4. inexcusablemente debe realizarse una campaña de caracterización de los materiales de la presa y de la geotecnia de la cerrada que soporte la aplicación de métodos de cálculo modernos que nos permitan evaluar el estado de seguridad de la presa;
5. en la estabilidad de una presa de gravedad antigua cobra especial relevancia el control de las subpresiones y;
6. disponemos de tecnologías adecuadas para la rehabilitación de los sistemas de control de subpresiones y para la regeneración de hormigones, algunas de las cuales fueron especialmente adaptadas para el caso de La Minilla.

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Background, planning and execution for rehabilitation of Valldalen and Vasstølvatn dams

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ABSTRACT: Valldalen dam is a rock-fill dam with moraine core, height 93 m and crest length 360 m. The spillway consists of a free flow structure, a shaft and a horizontal tunnel.

Vasstølvatn dam is a CFRD type, height 26 m and crest length 320 m with a free flow spillway structure.

"Norwegian water resources and energy directorate" (NVE), administer the regulations requiring that each Norwegian dam registered within Consequence classes shall be subject to regular re-evaluation each 15 year.

Re-evaluation of both Valldalen dam and Vasstølvatn dam revealed that the dams did not meet guideline criteria for damtoe drainage capacity, downstream slope erosion protection and dam crest freeboard.

In addition, the result of updated flood estimates at Valldalen dam also showed that the actual capacity of the spillway shaft and tunnel became exceeded.

This paper presents the background, planning of actions to be taken and execution of works to be performed in order to meet the guideline criteria.

1 INTRODUCTION

Valldalen dam and Vasstølvatn dam will be presented individually as there are differences in the planning and procedures versus NVE.

The first "Action plan for rehabilitation of Valldalen dam" presented to NVE was dated December 2004. A revised Action plan for rehabilitation of Valldalen dam presented to NVE was dated December 2005. Meetings and clarifications were held between NVE and NHP before a final decision on the actions to be implemented were reached.

Next stage in the communication with NVE was presentation of "Technical detailed plan" for the rehabilitation and a "Control program" to be implemented during the construction works for the rehabilitation, both dated September 2006.

Construction works started immediately after the Technical detailed plan and Control program was approved by NVE, i.e. autumn 2006, and continued through 2007 and was completed by autumn 2008.

The first "Action plan for rehabilitation of Vasstølvatn dam" presented to NVE was dated June 2008. A revised Action plan for rehabilitation of Vasstølvatn dam presented to NVE was dated January 2009.

Based on the experience gained during the planning process for Valldalen dam, the planning process for Vasstølvatn dam became more streamlined. However, minor adjustments had to be made during the construction as it always happens.

NVE's approval of the Action plan was received on May 26, 2009 and the construction works started immediately thereafter. The works were completed by October 2009.

Contractor for civil works at both dams was Skanska Norge AS.

Consultant for both dams was Sweco Norge AS.

2 VALLDALEN DAM

2.1 *Main data for Valldalen dam*

Rock fill dam with central core of moraine constructed in 1964–1965.

Dam toe foundation at elevation 656.0 masl.

Dam crest at elevation 749.0 masl.

Upstream slope 1:1.6, covered by rock blocks.

Downstream slope between foundation level and elevation 700.0 masl. 1:1.6, further slope 1:1.45 to crest elevation. Downstream slope consist of dumped and leveled rock materials from quarry.

Downstream slope before start of the rehabilitation.

Looking downstream from dam crest.

HRW, highest regulated water level:	745.0 masl.
LRW, lowest regulated water level:	665.0 masl.
Dimensioning Flood Level (Q_{1000}):	746.12 masl.
Probable Maximum Flood level:	746.73 masl.
Probable Maximum Flood:	647 m^3/sec.
Height of Valldalen dam:	93 m.
Length of dam crest:	360 m.

2.1.1 *Spillway arrangement*

Free flow weir, concrete structure, length 132 m.

Downstream of weir, excavated canal leading into 45° Shaft with cross section 17 m^2, 40 m long tunnel section with cross section 22 m^2 and 200 m long tunnel section with cross section 47 m^2.

2.1.2 *Other arrangements*

The power intake in the reservoir is located outside the dam area. Access to the power intake structure and gates is arranged for by road along the crest of the dam.

Bottom outlet and gates are located in rock just under the dam foundation. During construction of the dam, the bottom outlet was operated as diversion tunnel. Part of the bottom outlet tunnel system now form part of the spillway. Access for operation of the gates is arranged for by a separate tunnel.

2.2 *Pre-rehabilitation documentation*

In order to confirm the top level of the moraine core, excavation of test pits were done at or close to each of the settlement bolts installed at the dam crest. The elevation of each bolt was measured and confirmed including also documentation by photos of the different zones at the top of the dam crest.

To make sure that the conditions for the free flow spillway structure could be implemented for the flood release, a set of calculations was prepared as documentation for the capacity of the spillway canal, the shaft and the tunnel section of the spillway based on that the cross sections had to be increased. The existing spillway system had a capacity of approximately 350 m^3/s whilst the PMF (probable maximum flood) in the updated flood estimates was calculated at 647 m^3/s. As a result the spillway tunnel with a length of 200 m had to be increased from the existing 47 m^2 to 61 m^2, a shorter stretch, 40 m, had to be increased from existing 22 m^2 to 38 m^2, and the shaft, length 110 m, had to be increased from 17 m^2 to 30 m^2 including a trumpet lay out at the top increasing the cross section to 64 m^2.

To be able to decide the required grain size distribution for the different zones to be built in for downstream slope erosion protection, materials for testing was taken from the lower part of the downstream slope and sent for sieve analysis.

Mapping of the dam, i.e. the downstream slope and dam crest was done by using a type of scanning equipment. The map was intended to be used for documentation and calculation of filled-in volumes after completion of the works. A separate map was prepared for the area where the initial quarry was located, beneath HRWL within the reservoir.

An Area use plan, which includes all environmental aspects, was prepared ahead of the rehabilitation. The plan was sent to the municipal and province authorities. The authorities held public hearings and collected points of view from affected land owners and different NGO's in additions to their own requirements which then had to be implemented into a revised Area use plan.

2.3 *Stage wise development of Action plan for rehabilitation*

Based on the results of the pre-rehabilitation documentations, the initial Action plan for rehabilitation

included a minimum of downstream slope works with establishment of a 13 m high dam toe at the lower part of the downstream slope and a 2.5 m high erosion protection barrier along the right side of the foundation zone of the dam.

The plan also included the necessary excavation of the spillway canal, shaft and tunnel such that the conditions for the spillway structure was met to control the flood level rise without any risk for overtopping the moraine core. The actions were documented by spillway capacity calculations and geotechnical stability calculations for the dam body.

NVE did not approve the Action plan and required further documentation. NVE also made reference to the fact that Valldalen dam at earlier stage had experienced some incidents like sudden leakage with grayish water, sinkhole at dam crest, leakage alarm initiated by snow avalanche. The main missing documentation was related to NVE's requirement of a complete erosion protection of the downstream slope. In addition NVE pointed at some adjustments needed for the spillway capacity calculations.

A revised Action plan was prepared. In the revised plan it was still argued that the existing guidelines for rock fill dams did not specify a complete downstream slope erosion protection and updated calculations was prepared for documentation of the geotechnical stability. Regarding the other requirements from NVE, the revised plan was completed with updated documentations and calculation.

In the discussions that followed between NVE and NHP and involved consultants including third party verification, it was realized that NVE's requirement of a complete downstream slope erosion protection had to be implemented into the action plan. The Technical detailed plan and Control plan then was prepared taking this complete erosion protection into account.

Due to the lengthy approval process with NVE, NHP had initiated the tendering process ahead of the final approval in order to gain time. Through the discussions with the selected contractor, a new solution for the internal roads within the construction area was found. Instead of extending an existing road through the cottage village downstream of the dam, the contractor suggested to make a temporary road along the downstream right side of the dam.

As a result of the new internal road alignment and the new design, with a complete downstream slope erosion protection zone, the need for excavated rock had increased rather heavily, from an estimated volume of approximately 35,000 m^3 up to more than 140,000 m^3. This again led to further investigations for additional quarry locations to make sure that the necessary volume was available within reasonable distance from the dam. Consequently the Area use plan had to be revised for the second time and go through a new approval procedure.

NVE and NHP agreed that the rigging works could start in accordance with the approved original Area use plan even though the approval of the Action plan remained to be given.

2.4 *Start of construction works*

Mobilization took place in September 2006 and was aimed at getting the dam toe and as much as possible of the downstream erosion protection built in before the winter and snowfall set a stop for the season.

The new zones was designed with a total thickness of approximately 6.5 m consisting of a 3.5–4 m supporting rock zone and a 2 m thick outer rip-rap zone consisting of rock blocks, minimum diameter 0.65 m, placed by backhoe. It should also be mentioned that all humus layers on the existing downstream surface should be removed before placing the new zones. The supporting zone acted also as access for the trucks and backhoe when dumping and placing the rock blocks for rip-rap zone.

Instead of using set-out poles and traditional land survey equipment, the backhoe was equipped

Terrain "nose" location of additional quarry.

with GPS and laser which by computer connection gave the operator a "beep" signal when each of the rock blocks were in the correct position.

It was revealed rather soon that the quality of rock in the initial quarry had a lot of fissures and cracks which made it complicated to collect the expected volume of large blocks for the dam toe and rip-rap dam zone. Even though the drilling and blasting procedure was designed in order to get a large part of rock blocks, the loading and intermediate storing of materials, sorting out the rock blocks and reloading resulted in an unexpectedly low percent of useful rock blocks, less than 5%.

Still, the contractor almost managed to complete the dam toe, and the construction site was closed by December 21, 2006. In parallel to the dam works, excavation for increasing the cross section of the spillway tunnel, both sections, were completed before the site was closed.

2.5 *Organization at the construction site*

The dam owner's organization was limited to site manager who also acted as the head of the owner's control staff on behalf of an independent responsible control chief, certified according to NVE's guide lines and approved by NVE. This arrangement, having a site manager acting on behalf of an independent chief control person was discussed and agreed upon with NVE prior to the start of the construction works.

NVE even accepted that shift control was taken care of by university level students. This last acceptance came as result of that NVE realized that dam construction, supervision and related activities like utility company staff is in urgent need of recruitment as the aging among employees in this business in getting a problem.

The contractor's staff and employees at site consisted of a construction manager, two foremen, surveyor and machine operators for drilling, blasting, loading and operators of trucks, backhoes, wheel loaders etc.

Progress reporting meetings were held on weekly basis and project meetings on monthly basis. In these meetings the dam owner's representative responsible for all hydropower structures took part.

HSE, health, safety and environment, check rounds were held on regular basis. In order to prevent contamination from the construction site in case of oil leakage from the machines and equipment or from the quarries, barriers were placed across the water flow downstream of the construction site. Samples of water was taken on regular basis and sent for analyses as part of documentation.

Occasional visits by the chief control person took place in combination with taking part in either a weekly meeting or a project meeting. The chief control person received an updated weekly report including pictures every Monday and monthly progress reports. After approval of the monthly reports they were sent to NVE.

Supervision inspection.

Inspections by personnel from NVE, dam safety section and environmental section, took place on average twice each season.

A program for surveying all installed deformation and settlement bolts at the moraine core, at the dam crest and the dam slopes by the start and end of each season was implemented.

As part of the control program, material tests of the built in supporting zone should be taken at least once each season and sent for sieve analysis as part of the documentation versus NVE. According to NVE guidelines a complete report covering all activities for all works shall be submitted for each season.

2.6 *Construction works challenges 2007*

Due to the rock quality in the initial quarry discovered in 2006, it was decided to implement a potential third quarry location into the Area use plan early in 2007. The location of the third quarry would be in a distance of approximately 16 km away from the dam. In the meantime discussions were started with NVE's environmental representative and the municipal authority in order to get approval to access the second quarry location which partly was located above HRW and partly in the reservoir. Discussions and approval procedures for the part above HRW went on over the winter and access to the second quarry above HRW was accepted shortly after start of dam season 2007.

Access to the construction site was kept open in order to continue the excavation for increasing the cross section of the remaining part of the spillway, i.e. the shaft, which was completed by the 8th of May 2007.

Excavation in the second quarry part in the reservoir started as early as the end of February 2007. The reservoir was at low level at the start of the 2007 season and the restrictions implemented for keeping it low allowed for excavation until primo June 2007. Almost all rock from this part was brought to temporary storage.

The second quarry part above HRW involved relocation of a shorter stretch of a local road leading into a national park. The road had to be closed temporarily before each blasting took place.

The dam season 2007 started the same day as the spillway shaft excavation was completed, i.e. May 8. The stress factor related to the operation of the quarry in the reservoir urged the contractor to spend most of the equipment for bringing out as much materials as possible for the reservoir before the run-off from the snowmelt filled the reservoir.

Another challenge came to the surface when it was discovered that one of the supporting beams for the spillway structure was destroyed. A 12–13 m long concrete beam had suffered damages as result of lack of protection when blasting at the inlet to the spillway. The reservoir level continued to rise and time became too short for replacement by a new concrete beam. Therefore a local mechanical contractor was engaged in order to come up with a temporary extra support by introducing a steel beam which had to be placed by a heavy mobile crane. The extra support and the damaged concrete beam proved to handle the situation which in fact included a period with spill over the concrete structure. A permanent new prefab concrete beam was installed and safely connected in both ends by rock bolts and embedded in concrete in the spring of 2008.

Approval procedure for the potential third quarry was still in process and continued being undecided through the whole 2007 season.

Fortunately the second quarry location showed to give the expected percentage of large rock blocks. The part in the reservoir reached a rock block percentage of 24, while the part above HRW reached a rock block percentage of close to 35. From the quarry part above HRW the sorting could be done directly in the quarry for transportation either to temporary storage or directly on to the dam whenever that became the best option. The optimistic view at the end of the 2007 season was that the second quarry could meet the total need for rock and thereby the third quarry hopefully would become unnecessary.

The construction site was closed on 20 November 2007. The total volume of rock built in during 2006 season and 2007 season amounted to 112.500 m^3.

Additional quarry and relocation of road.

2.7 *Continued construction works challenges 2008*

During the winter 2007–2008 a lot of snow hampered the start of the dam season and the contractor spent two week clearing the access road between the public road and the dam site and another two week removing snow at the construction site and the dam body.

Start of the construction works took place on 21 April 2008 and from 5 May the contractor decided to work day shift with the rip-rap zone and night shift with the supporting zone in order to get out as much rock as possible from the quarry part in the reservoir. This effort showed to be a wise decision for both the contractor and NHP as the need for the third quarry disappeared.

The discussion related to determination of built in volumes had started already during the 2007 season and continued through the whole 2008 season. The contractor and NHP had rather different opinions on the basis for the calculations with reference to the scanned maps taken before the works started. Two major factors were affecting the calculations and results.

NHP had decided that all loose materials along the dam foundation should be removed and taken to spoil heap before placing the new zones. It showed that this volume became much larger than expected and had of course to be compensated by extra volume in the new zones. The removed volume was well documented by the contractor by GPS mapping and additionally by counting the number of truck loads. NHP in the end found no reason to question the GPS surveying figures presented by the contractor in spite of the additional cost.

The next volume discussion was related to removal of the humus layer on the existing dam slope and consequences for compensation by extra thickness of the new supporting layer. In this discussion the contractor insisted that due to the removal of humus, the compaction of the new supporting zone

would penetrate into the existing dam body and even some of the existing dam body would be subject to extra compaction and thereby make room for additional new materials to be built in. NHP never accepted this type of argument and the discussion finally was closed by an agreement which included a lump sum to the contractor as premium for finalizing the works within the 2008 season including compensation for the extra work related to quarries.

According to conditions set by NVE's environmental representative, the spoil heap consisting of over break materials and all excavated materials which was not found of acceptable quality to be built into the new zones should not exceed a predecided volume and height in the terrain. Due to the missing quality of the initial quarry, extra volume was placed in the spoil heap and NHP had no possibility to finalize the lay-out of the spoil heap within the given conditions at the closure time of the works. In the meantime the municipal authorities had approached NHP and asked if they were allowed to use some of the excess volume for construction of a parking area in the vicinity of the cottage village. Local road authorities also showed interest in the excess volume as it can be used for crushing and road maintenance in the years to come.

A final decision for disposal of the excess volume was not taken and NVE accepted a temporary delay of finalizing the spoil heap lay-out. The contractor became therefore released from the contract obligation to make the final lay-out.

All other activities related to environmental aspect, especially the treatment of all areas used during construction period for rigging, camp and offices, temporary roads and excavation along dam foundation were rounded off and re-cultivated with selected seeds of local type. The final activity was to clean the complete downstream rip-rap slope by air blowing in order to prevent that loose material could cause problems and accidents in regular future inspections.

At the time of closing the site the total built in volume during 2008 amounted to approximately 42,500 m^3, bringing the total built in volume to 155,000 m^3.

During the 2008 season NHP engaged the contractor also to include some extra works outside the original contract such as:

Construction of a concrete slab as access into the reservoir for setting out and taking up boats, see picture below.

Placing asphalt layer at a section of the steepest part of the access road to the cottage village and to the dam.

Drainage of water flowing from the terrain into the leakage arrangement in order to have the flow passing by on the outside of the leakage arrangement.

Leading the leakage water from the leakage arrangement through a 100 m long pipe in such a way that snow avalanches no longer will create the leakage water level to build up and generate false alarm.

A separate limited leakage was discovered with the reservoir level at approximately 732 masl. i.e. from 13 m below HRW. The drainage flow was very regular however it became diverted into a specific course for inspection and observation in case the flow increases over time.

2.7.1 *Lessons learned*

The importance of having a proper definition of basis map and specification on contractor's responsibility to verify basis and how to measure built in volumes

By entering 2010, NVE has implemented a new regulation for all structures in watercourses, i.e. rivers and reservoirs. The regulations are approved by the Norwegian Ministry of Oil and Energy. The new regulations have been under preparation for some time and NVE has of course had the regulations in mind when arguing with dam owners about the requirement for downstream slope protection on existing dams, so also for the Valldalen dam.

In retrospective, at present stage there is no doubt that it is was correct decision to implement

Spoil heap in front for future road maintenance.

a complete erosion protection zone on the whole downstream slope at Valldalen dam. If that had not been done during 2006–2008, yet another construction period would be necessary to fulfill the new regulations with all related extra works and not to mention extra cost.

3 VASSTØLVATN DAM

3.1 *Main data for Vasstølvatn dam*

Concrete face rock fill dam constructed in 1965–66.
Dam toe foundation at elevation 729.0 masl.
Dam crest at elevation 755.0 masl.
Upstream slope 1:1.25, concrete face.
Downstream slope 1:1.3. Downstream slope consist of dumped and leveled rock materials from tunneling and quarry.
HRW, highest regulated water level: 753.0 masl.
LRW, lowest regulated water level: 732.5 masl.

Downstream slope and terrain after rehabilitation.

Dam crest after rehabilitation.

Dimensioning Flood Level (Q_{1000}): 753.86 masl.
Probable Maximum Flood level: 754.08 masl.
Probable Maximum Flood: 284 m^3/sec.
Height of Vasstølvatn dam: 26 m.
Length of dam crest: 322 m.

3.1.1 *Original spillway arrangement*

Free flow weir, concrete gravity dam, length 110 m.

Downstream of weir the flood flows into an excavated canal along dam toe into original river course at approximately center of dam.

3.1.2 *Spillway arrangement after rehabilitation*

Free flow weir, concrete gravity dam, length 110 m.

Downstream of weir the flood flows into a complete new excavated canal then enters a drop of approximately 10 m and passes through two small pools before entering the original river course.

3.1.3 *Other arrangements*

The power intake in the reservoir is located outside the dam area.

Bottom outlet and gates are located in rock just under the dam foundation. During construction of the dam, the bottom outlet was operated as diversion tunnel. The gatehouse sits at the dam crest located just between the CFRD and the spillway.

3.2 *Pre-rehabilitation documentation*

To be able to decide the required grain size distribution for the zones to be built in for downstream slope erosion protection, materials for testing was taken from the lower part of the downstream slope and sent for sieve analysis.

Mapping of the dam, i.e. the downstream slope and dam crest was done by surveying equipment. The map included the area where the quarry was located which was to be excavated in order to facilitate a new spillway away from the dam toe.

Vasstølvatn dam site during flood before start of rehabilitation works.

Existing spillway. Rock bench to be excavated.

Inlet to old spillway canal.

An Area use plan, which includes all environmental aspects, was prepared ahead of the rehabilitation. The plan was sent to the municipal and province authorities. The authorities held public hearings and collected points of view from affected land owners and different NGO's in additions to their own requirements which then had to be implemented into a revised Area use plan.

3.3 *Stage wise development of Action plan for rehabilitation*

Based on the results of the pre-rehabilitation documentations, the Action plan for rehabilitation included new zones on the downstream slope with a total thickness of approximately 6.5 m consisting of a 3.5–4 m supporting rock zone and an outer rip-rap zone consisting of rock blocks, minimum diameter 0.65 m, placed by backhoe. In addition, establishment of a 2 m high dam toe along the foundation between the area for the original river course and the spillway. The plan also included the necessary excavation of the new spillway canal integrated in the quarry. Relevant geotechnical stability calculations formed part of the documentation.

In order to meet updated guidelines for registration of leakage, three new leakage arrangements for continuous registration were implemented in the Action plan.

NVE did not approve the Action plan and required further documentation regarding the dam crest volume capability to resist extreme loads and the height of the dam crest above HRW. NHP faced two alternatives, either widening the new zones or rising the dam height.

A revised Action plan was prepared meeting the requirements presented by NVE. The total thickness of the new zones was widened to 8.75 m and a 1.5 m heightening of the dam crest was implemented and approval procedure went on. In the meantime the Areal use plan was approved.

While waiting for the NVE approval, NHP had initiated the tendering process in order to gain time. Skanska became the selected contractor once again. NHP agreed to let the contractor mobilize and start rigging and preparatory works according to the approved Area use plan while waiting for NVE's approval of the Action plan.

A Control plan was prepared and sent to NVE.

3.4 *Organization at the construction site*

The organization at Vasstølvatn dam became more or less identical to the Valldalen organization.

The dam owner's organization was limited to site manager who also acted as the head of the owner's control staff on behalf of an independent responsible control chief.

NVE accepted once again that shift control was taken care of by university level students. The same students came back and got another season with valuable experience.

The contractor's staff and employees at site consisted of a construction manager, one foreman, surveyor and machine operators for drilling, blasting, loading and operators of trucks, backhoes, wheel loaders etc.

Progress reporting meetings were held on weekly basis and project meetings on monthly basis. In these meetings the dam owner's representative responsible for all hydropower structures took part.

HSE, health, safety and environment, check rounds were held on regular basis. In order to prevent contamination from the construction site in case of oil leakage from the machines and equipment or from the quarries, barriers were placed across the water flow downstream of the construction site. Samples of water was taken on regular basis and sent for analyses as part of documentation.

Occasional visits by the chief control person took place in combination with taking part in either a weekly meeting or a project meeting. The chief control person received an updated daily

report including pictures every morning and weekly report including pictures every Monday plus monthly progress reports. After approval of the monthly reports they were sent to NVE.

Three inspections by personnel from NVE, dam safety section and environmental section took place.

A program for surveying all installed deformation and settlement bolts at the dam crest and the dam slopes by the start and end of the season was implemented.

As part of the control program, material tests of the built in supporting zone was taken and sent for sieve analysis as part of the documentation versus NVE. According to NVE guidelines a complete report covering all activities for all works was prepared and submitted to NVE after completion of the works.

3.5 *Construction work challenges*

Approval of the Action plan by NVE was received on 26 May 2009 and the dam works started immediately after. As mentioned in previous chapter, the new zones was designed with a total thickness of 8.75 m consisting of a 6.5–6.75 m supporting rock zone and a 2 m thick outer rip-rap zone consisting of rock blocks, minimum diameter 0.65 m, placed by backhoe.

As for Valldalen dam, the backhoe was equipped with GPS and laser which by computer connection gave the operator a "beep" signal when each of the rock blocks were in the correct position instead of using set-out poles and traditional land survey equipment.

Based on the experience related to that the quality of rock in the initial quarry at Valldalen dam had a lot of fissures and cracks which made it complicated to collect the expected volume of large blocks, the lay-out of the quarry at Vasstølvatn had taken this into consideration and was designed for a total excavation volume of more than 90.000 m^3. Fortunately the rock quality showed to be of excellent quality and the percentage of rock blocks was much larger than expected. The challenge then became to limit the excavation to fit with the actual needed volume. Adjusting the quarry lay-out became hard to follow up as the reporting of filled in rock and estimation of remaining volume to be built in was delayed due to lack of proper surveying. After completion of the construction works at the dam, which ended at a total volume of 66.600 m^3, still a lot of excess excavated rock was available and had to be placed somewhere in the area.

Inspection by NVE and supervision. Rock blocks closing inlet to old spillway canal.

By a combination of terrain leveling and backfilling along the original spillway canal in combination with environmentally actions to round off all areas used for camp, offices, temporary roads etc. the excess rock became integrated into the terrain and was re-cultivated by local type seeds. The final solution of terrain lay-out was discussed and agreed with the environmental representative from NVE which has approved the result afterwards.

As a result of the extra works related to the excess excavated volume, the contractor and NHP disagreed on which unit prices in the BoQ should be used in the set-up for the invoice for the final payment. The discussion finally was closed by an agreement which included a lump sum to the contractor as premium for finalizing the works within the 2009 season.

Due to the relatively small reservoir in Vasstølvatn and the fact that the existing spillway canal would be put out of operation during the construction time while the new spillway canal would not be established until finalization of the excavation, NHP had to set limitation for the operation of the reservoir during construction in order to avoid overtopping the spillway. The reservoir was lowered 10 m at the start of construction and kept at that level or at lower level through the construction period. In case of emergency like heavy rainfall or stop of the power plant that took water from the lowered reservoir, a procedure for opening the bottom outlet was implemented. Fortunately no emergency situation occurred and no water had to be diverted through the bottom outlet.

As result of the 1.5 m heightening of the dam crest an access problem occurred at the entrance of the bottom outlet gate house located between the CFRD and the spillway. The original access to the gate house is located at the original dam crest level and the new access road on the dam crest to the gate house became at a 1 m higher level. The problem became solved by introducing a stone stepped section just at the end of the access road.

During the construction period NHP engaged the contractor also to include some extra works outside the original contract such as:

Construction of a concrete slab as access into the reservoir for setting out and taking up boats identical to the arrangement at Valldalen dam.

Dam and new quarry with new spillway canal after completion and backfill of local soil materials.

Old quarry and concrete slab for boats, above.

Placing part of excess rock in an old quarry, rehabilitation of terrain and re-vegetation of the surface.

Relocation of a shorter section of an access road passing by the Vasstølvatn dam to another dam including installation of new culverts for diversion of a local creek.

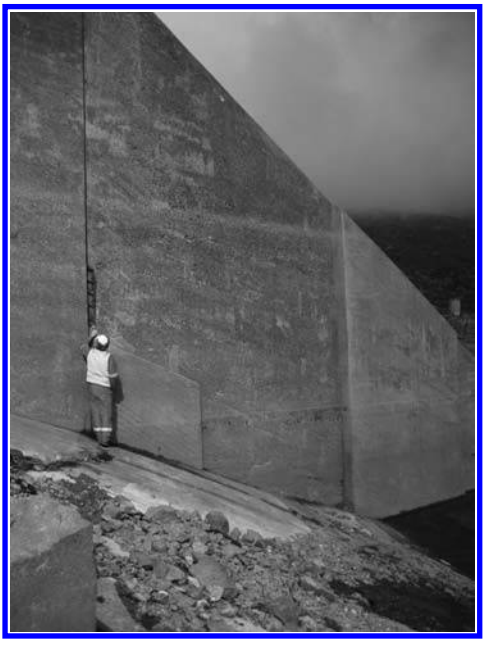
Minor repair of concrete structures related to the concrete gravity spillway and the gate house, below.

3.5.1 *Lessons learned*

The advantage of letting the contractor approve the maps that forms the basis at the start of the works and from then on be responsible for updating maps after completion of the works. As a result the final discussion on how much rock has been excavated and how much rock has been built in the dam becomes much simpler.

The importance of having a proper and regular routine for surveying of the status of filled in rock, remaining volume to be filled in, rock in temporary storage and status of excavation in the quarry at given stages during the progress of the project.

The importance of having a skilled and trained contractor with a minimum of, but necessary, equipment, machines and well experienced staff, including guaranteed access to back-up machines and personnel.



Dam crest after completion of the rehabilitation works.

Rehabilitation done in one season. Normal situation restored.

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Acondicionamiento de la presa de Puente Porto (Zamora)

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ABSTRACT: Puente Porto is a dam composed of masonry, concrete cut off and masonry buttresses. It was completed in 1953. Its foundations are 27,3 m. high and the crest length is 325 m. Located in the province of Zamora is a key element in the scheme of integral utilization of the river Tera. When 50 years have passed since the disaster in the Vega de Tera dam, twin of Puente Porto dam, and located just 8 kms from the former, the conditioning of this dam, undertaken by Endesa, adquires, if anything, more relevance.

1 DESCRIPCIÓN DE LA PRESA

La presa de Puente Porto es una presa de contrafuerte de los años 50 situada en un Parque Natural cerca del lago de Sanabria en la provincia de Zamora. Es una presa mixta de contrafuertes de mampostería con pantalla de hormigón y espaldón de escollera, con una altura desde cimientos de 27,30 m y una longitud de coronación de 325 m.

Esta presa es gemela de la presa de Vega de Tera, ubicada a tan sólo 8 km, tristemente famosa por su rotura en enero de 1959 causando el mayor desastre debido a una rotura de presa en España. Esto ha hecho que este acondicionamiento, emprendido por ENDESA, haya cobrado aún mayor relevancia.

Una semana después de la rotura y en cumplimiento de la Orden de 15 de Enero de 1959, la Dirección General de Obras Hidráulicas estimó necesario proceder a la reforma de esta presa añadiendo una escollera. Luego se ha demostrado que esto no tiene ningún efecto puesto que para que empiece a trabajar la escollera se tendría que deformar excesivamente la pantalla de hormigón; la enorme diferencia de módulos de elasticidad entre el hormigón y los diversos materiales que conforman el resto de la presa hace que bajo un mismo esfuerzo las deformaciones sean distintas, y que la pantalla de hormigón alcance el límite último de rotura antes que su deformación transmita la tensión del empuje del agua a la escollera vertida.

Desde 1995 el equipo de vigilancia y auscultación de ENDESA ha venido alertando sobre la aparición de nuevas filtraciones (600 litros/minuto) coincidiendo con el incremento de cota en el embalse. Se realizaron campañas de inyecciones (1996,1997 y 1999) para sellar las entradas y se limita el nivel al 45% de capacidad, donde se certifican unas filtraciones admisibles.

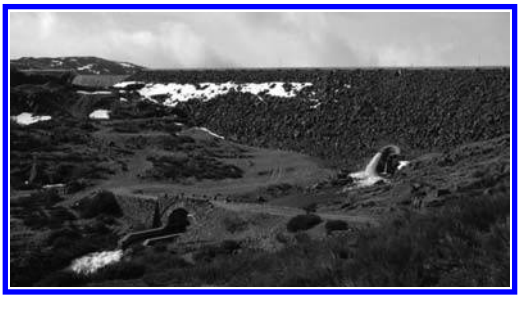

Figura 1. Vista de la presa de Puente Porto desde aguas abajo.

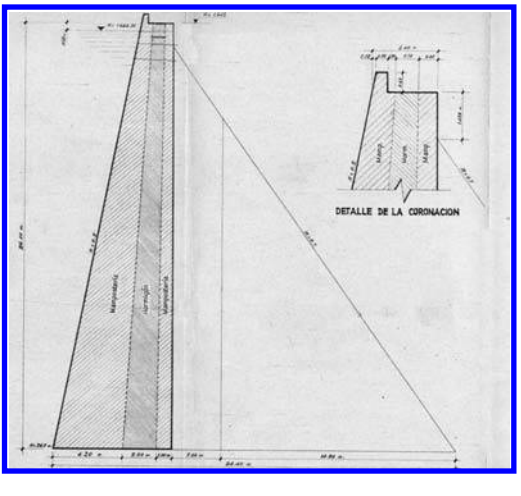

Figura 2. Sección transversal por un contrafuerte antes del vertido de la escollera aguas abajo.

ENDESA analizó la posibilidad de recuperar la capacidad original del embalse dentro de las máximas condiciones de seguridad posible junto a otra serie de actuaciones de modernización, de cara a la inminente implantación de los Planes de Emergencia de todas las presas del sistema.

2 DIAGNOSIS ANTERIOR AL ACONDICIONAMIENTO

En 1995–96 se decidió encargar una diagnosis de la situación Hidráulico-Estructural de la presa a Tecnología Hidráulica S.L.: los profesores D. Jose Antonio Baztán, catedrático de Presas en la Universidad Politécnica de Madrid y D. Antonio de Santiago, con el fin de determinar el estado de las presas del Sistema Tera en relación con su estructura principal según diseño de proyecto y su comportamiento en presencia de distintas solicitaciones hidráulicas. Este estudio tenía por objeto dar respuesta a algunos interrogantes referentes a Puente Porto, a la luz de Reglamentos, Instrucción y Guías Técnicas de Presas.

Para realizar este estudio y siguiendo los consejos de la diagnosis, se contrató una campaña de sondeos y posterior testificación para definir con rigor las características reales y actuales de los diferentes materiales que forman la presa, se trataba de determinar el estado del hormigón de la pantalla, el mampuesto y la roca de cimiento. La idea inicial era construir una pantalla impermeable e inocua que impidiera el paso de aguas al interior del cuerpo de presa permitiendo recuperar la capacidad máxima inicial del embalse.

El resultado de la campaña de sondeos fue inesperado. Los taladros mostraban un material en mal estado y alterado con una recuperación de testigos ínfima. En la fotografía se puede observar como en 8 metros perforados se recuperaron 1,80 metros. Ante la ausencia de documentación técnica fiable se aceptó como hipótesis constructiva el mismo perfil que la presa de Vega de Tera; se demostró que el sistema constructivo era el mismo aunque no se reproducía en los testigos: El problema estaba en la lixiviación por las aguas tan puras que habían hecho que la pantalla de hormigón se hubiese debilitado. También hay que tener en cuenta la calidad de los materiales, la altitud y cantidad de nevadas al año y los accesos.

Figura 3. Ensayo de caracterización geométrica y mecánica mediante sondeo.

En el resultado de la Diagnosis se mencionó la necesidad de buscar una **solución definitiva** al problema de las filtraciones.

3 PROYECTO DE ACONDICIONAMIENTO DE PUENTE PORTO

Con estos datos, ENDESA encargó a Tecnología Hidráulica, la redacción del proyecto de adecuación, cuya recomendación fue una impermeabilización continua desde profundidades semejantes a la altura de la presa en la cimentación hasta la coronación y un rebaje del antiguo aliviadero para reducir las solicitaciones de la presa asegurando de esta manera la estabilidad del conjunto, a la vez que hacer cumplir con la normativa actual.

Para impermeabilizar de forma continua desde el cimiento hasta la coronación se propuso realizar una campaña de inyecciones de consolidación e impermeabilización del cimiento, poniendo especial énfasis en la interfase impermeabilización-inyecciones. Además, la impermeabilización contemplaba tratar el paramento de aguas arriba para evitar fugas y el progresivo deterioro del hormigón restante.

Con este planteamiento, ENDESA abordó la ejecución de las obras, con una visión holística, contemplando el **proyecto como un todo**. Acudió para la realización de determinadas partidas a empresas locales (SUCASPI, FERPE, TRES CASTILLOS, MANCEÑIDO y GESNA MEDIOAMBIENTAL) y confió la coordinación general de los trabajos, sobre todo en materia de Prevención, Calidad y Medio Ambiente a HCC .Además, ésta se hizo cargo desde el diseño y la colocación de los andamios, hasta los trabajos de inyección del cimiento y el control de la impermeabilización.

4 LICENCIAS MEDIOAMBIENTALES

La presa de Puente Porto se encuentra afectada por las siguientes figuras de protección: Parque Natural del Lago de Sanabria, ZEPA ES4190009, LIC ES 419105 y LIC ES4190067 Riberas del Río Tera y afluentes. Además, el embalse de Puente Porto

Figura 4. Acceso nuevo construido para evitar usar el antiguo puente romano.

está declarado como Zona Húmeda catalogada. Todo esto hizo que en el acondicionamiento de la presa se vieran implicados medioambientalmente el Ministerio de Medio Ambiente (Servicio de Vigilancia de Presas), la Confederación Hidrográfica del Duero, la Junta de Castilla y León (Dirección General del Medio Natural) y el Parque Natural del Lago de Sanabria y alrededores.

Se establecieron como requisitos a la licencia medioambiental para el acondicionamiento de Puente Porto, mantener la pista de acceso en el estado existente, lo que obligaba utilizar medios de transporte y maquinaria adecuados a las condiciones de la misma, la restauración de los viales y pasos de agua dispuestos sobre el cauce del río ubicado aguas abajo de la presa una vez terminadas las obras, mantener los caudales libres aguas abajo de la presa durante el periodo de ejecución de las obras, regenerar de forma vegetal y paisajista el lugar de depósito definitivo del material excedente de la ejecución de la obra, mantener la calidad de las aguas y no afectar a la comunidad biótica y un plan de vigilancia ambiental llevado a cabo por una asesoría técnica medioambiental independiente del promotor durante la ejecución de las obras y durante un año una vez terminada ésta.

Por último, como medidas complementarias de restauración del entorno se contempló la ejecución de labores de mejora del hábitat de la perdiz pardilla mediante desbroce selectivo de matorral en una superficie de 30 hectáreas y la siembra de cereal en zonas previamente desbrozadas sobre una superficie de 45 hectáreas.

5 REBAJE DEL LABIO DEL ALIVIADERO

En el proyecto de acondicionamiento de la presa de Puente se recomendó rebajar la cota del aliviadero 5,48 metros. Las nuevas condiciones de explotación (el rebaje del aliviadero y la disminución de la cota NMN) consiguen absorber todas las avenidas con periodo de retorno inferior a 5000 años y laminar la Avenida de los 10.000 años (Avenida Extrema según el Reglamento) desaguando solamente 35 m^3/s con una sobreelevación de la lámina de agua de apenas medio metro sobre el labio del aliviadero.

Esta reducción de la cota de explotación normal en tan sólo 1,25 metros ha permitido disminuir la cota del labio del aliviadero y tener sólo un decrecimiento de la garantía anual del 1,3% (además de como se dijo anteriormente laminar la avenida de los 10.000 años).

Con la nueva cota de explotación normal se ha logrado, bajo las hipótesis de carga que establece el Reglamento, que la presa carezca de tracciones (por lo que no se producirán aperturas entre mampuestos), que las compresiones máximas no pasen de 8 kg/cm^2, que las tensiones cortantes no superen los 2 kg/cm^2 y que presente una amplísima seguridad al deslizamiento, y en consecuencia, a la rotura por cizallamiento de su base, que fue una de las causas de la rotura de la estructura gemela de Vega de Tera (En el peor de los casos, la seguridad al deslizamiento de la solución que se ha realizado representa un aumento superior al 250% respecto a la seguridad requerida).

La solución adoptada, además de conllevar un recorte del aliviadero y nuevo perfil del mismo consistía en el tratamiento de los estribos laterales de hormigón del aliviadero en la zona descarnada por la demolición, la creación de un cuenco amortiguador en roca con azud final de hormigón de un metro para garantizar el calado conjugado de la curva de remanso generada en el vertido y con ello conseguir que el vertido transcurra en régimen lento por el río Segundera (aguas abajo de la presa), y la excavación de alimentación aguas arriba del vertedero para garantizar cotas y caudal de vertido.

Figura 5. Estado final del aliviadero una vez rebajado.

6 DIQUE AGUAS ARRIBA DE LA ZONA DE TRABAJO

Para poder trabajar en el paramento de aguas arriba sin tener que hacer un desembalse completo, y a petición de la Dirección General de Medio natural, de la Consejería de Medio Ambiente de la Junta de Castilla y León, se construyó un dique inmediatamente aguas arriba de la presa con objeto de dejar solamente en seco esa pequeña parte del embalse.

Este dique se realizó con materiales de la zona extraídos de la banda árida del embalse en una superficie tal, que sólo suponía una modificación suavizada de dicha zona, sin perjuicio de un terraplenado final de alisado a modo de restauración. La anchura de coronación del dique fue de 3'0 m como máximo ya que así se permitía el uso de pequeñas retroexcavadoras adecuadas para el arrastre y colocación del material. Este tipo de dique de tierra necesitaba de un núcleo impermeable central que estuviese cubierto por zonas del material existente en la zona y que fuese considerablemente más permeable y cuya función era cubrir y soportar dicho núcleo. En el caso que nos ocupa, la ejecución del dique se realizó en tres fases: construcción de la mitad del dique en sentido longitudinal con material grueso y heterogéneo, tendido y fijado de una lámina plástica que cubriese con suficiente margen el talud aguas arriba de la parte de dique construida y relleno del dique hasta alcanzar su anchura total y cubriendo la lámina que sirve de elemento impermeabilizante.

Además, como medidas medioambientales referidas al movimiento de tierras, se construyó una poza de amortiguación aguas abajo de la presa que atenuase la energía del agua y aumentase las posibilidades de sobrevivir de los peces que sean arrastrados en cualquiera de las maniobras que habitualmente se realizan en explotación normal a través de los desagües, y se construyeron remansos de sedimentación de finos en el cauce del río aguas abajo de la presa creadas con un pequeño muro en escollera de una sola fila de rocas en alzado consiguiendo con ellas reducir la velocidad del agua en descenso y proporcionando la sedimentación de los elementos más finos y el aclaramiento del agua.

Figura 6. Construcción del dique que permitiría trabajar en seco en la presa.

7 INYECCIONES DE IMPERMEABILIZACIÓN Y CONSOLIDACIÓN DEL CIMIENTO

La impermeabilización continua que se recomendó en el proyecto de acondicionamiento de la presa de Puente Porto requería realizar una pantalla de inyecciones con el fin de consolidar el terreno. Para realizar esta pantalla de inyecciones se realizaron previamente ensayos geofísicos y de tipo Lugeon con el fin de determinar la profundidad óptima para perforar. El ahorro con este tipo de ensayos, muy económicos, llegó a ser del 40%.

Para realizar esta evaluación mediante ensayos geofísicos, una vez descubierta la cimentación de la presa y su contacto con el terreno natural, y diseñada una red de inyecciones teórica se comenzó una campaña de perforaciones desde los pozos de

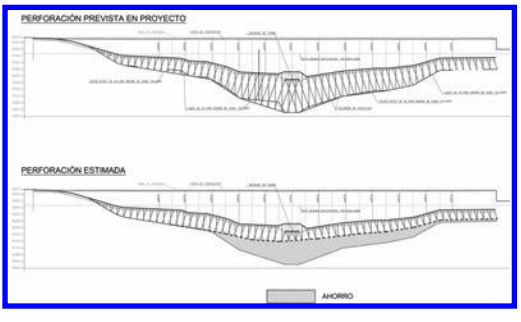

Figura 7. Ahorro conseguido mediante diagnóstico por ensayos geofísicos.

Figura 8. Pozos entre contrafuertes desde donde se hicieron los taladros.

inspección (espacios entre contrafuertes de la presa original) y se estudió la dirección apropiada de los taladros de inyección e identificó las zonas puntuales de filtraciones surgentes. Además, así se confirmaron las direcciones y buzamientos observados en la superficie y utilizados en el estudio inicial. Estos taladros servirían como puntos de inyección durante la campaña.

La ubicación elegida para los taladros de prospección fue cada dos pozos no coincidentes con juntas de construcción y con una inclinación de 5º hacia el embalse. La profundidad de cada uno de ellos se corresponde con la altura de la presa desde el fondo de cada pozo y el diámetro de perforación fue de 76 mm para permitir la máxima recuperación de testigos, la utilización de sondas y las inspecciones con cámara de televisión.

El sistema utilizado permite registrar de manera continua una serie de variables dependientes de las características de los sensores de cada sonda. Estos sensores son básicamente: para medir la temperatura del agua de la columna en el sondeo identificando zonas de aporte y sumideros, relacionadas con la concentración de sólidos disueltos en el agua y para determinar y cuantificar los caudales de entrada y salida del agua del taladro por zonas. La interpretación de los datos obtenidos permitió la evaluación precisa de los posibles puntos de discontinuidad y la importancia de las fisuras antes de realizar su inyección.

Según la definición geométrica de los taladros de inyección y a la vista de los testigos de la perforación, de la inspección con la cámara de TV y de los ensayos geofísicos, se preparó la programación de la inyección, que determinó el orden de inyección de los taladros, la longitud de los obturadores, la relación agua:cemento de la lechada, los posibles puntos de fuga y precauciones a adoptar si se producen; los consumos máximos admisibles por taladro, etc.

Se colocaron medidores de desplazamientos para controlar los eventuales movimientos de la estructura, en juntas, retomas de hormigonado y fisuras. Además, durante la inyección se llevaron a cabo trabajos de topografía en coronación.

Por último, una vez obturados todos los taladros, se realizó una inyección de agua a baja presión, anotando todas las incidencias, como comunicaciones entre taladros, caudales admitidos, fugas al exterior, etc.

Según requisito para la licencia medioambiental de mantener la pista de acceso en el estado existente, lo que obligaba utilizar medios de transporte y maquinaria adecuados a las condiciones de la misma, no se pudo usar una central grande de preparación de cemento, ni poner una red de circulación y una red de vuelta de la lechada. Se optó por ir a minicentrales de pequeñas producción

Figura 9. Trabajos de perforación sobre el plinto.

Figura 10. Trabajos de inyección para consolidar el cimiento.

(se esperaban pocas admisiones). Se trabajó hasta con 4 centrales a la vez. La ventaja de este tipo de central es que está informatizada y te permite controlar presiones mínimas y máximas. La otra ventaja es que los desechos generados son mínimos.

8 IMPERMEABILIZACIÓN DEL PARAMENTO DE AGUAS ARRIBA

Siguiendo con la impermeabilización continua recomendada en el proyecto de se impermeabilizó el paramento de aguas arriba mediante la colocación en superficie de una serie de láminas unidas entre sí.

Primero se colocaron unos rastreles y sobre esos rastreles se colocó la primera capa, una lámina drenante con masilla bituminosa incorporada para protección e impermeabilización complementaria de elementos a gran profundidad con doble propósito: dejar circular tanto agua como vapor por la parte de atrás y evitar el punzonamiento, ya

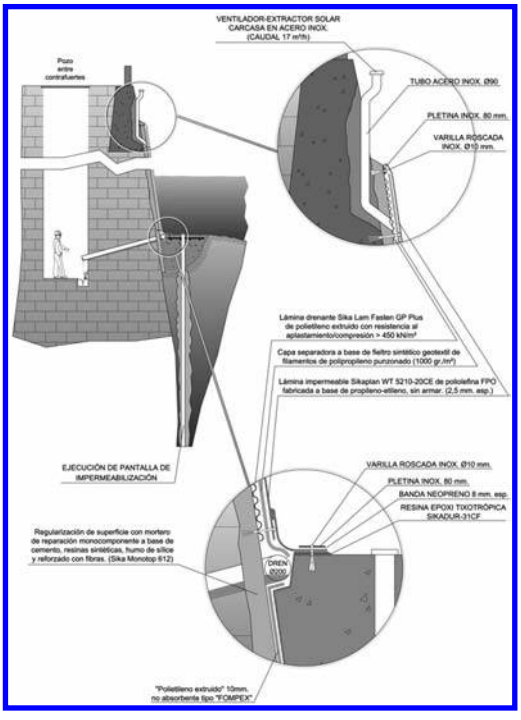

Figura 11. Detalle de la lámina, su drenaje y la pantalla de impermeabilización.

que no se hizo ningún tratamiento superficial del mampuesto. Sobre ello se colocó además una capa separadora a base de fieltro sintético geotextil de filamentos de polipropileno punzonado y ligado por termopresión, con una dotación de 1000 gr/m^2, colocado sobre el paramento en sentido vertical, y anclado mecánicamente. con objeto de reforzar la capacidad antipunzonamiento del conjunto. Estos rastreles perfectamente sellados constituyen zonas estancas que permiten si se produce una rotura de la impermeabilización localizar dónde está de modo aproximado.

Una vez colocada la lámina drenante y el geotextil se empezó a colocar la lámina de poliolefinas flexibles (FPO).

Estas láminas de poliolefinas FPO fueron fabricadas, a mediada bajo los criterios de la dirección, en base propileno-etileno, y cuyas ventajas frente al PVC tradicional son numerosas. Este material no ocasiona ningún tipo de perjuicio en contacto con el agua y es 100% reciclable pudiendo volver a fundirse de nuevo adquiriendo las propiedades iniciales (Membrana 100% ecológica libre de plastificantes, metales pesados y piroretardantes halógenos, se puede incinerar al final de su vida útil después de pasar varios procesos de reciclado, está Clasificado al fuego B-2, tiene mayor resistencia

Figura 12. Trabajos de colocación de las láminas impermeabilizantes en altura.

física 1,8 mm FPO = 2.2 mm PVC-P y también mayor resistencia al envejecimiento).

Otra de las principales características de esta lámina de poliolefinas es que el sistema de anclado al paramento se realiza, a diferencia de otros sistemas sin necesidad de tensado del material y que la lámina es termosoldable, realizándose las uniones entre paños de forma automática.

Con el fin de evitar las condensaciones se instaló en la parte superior del trasdós un original sistema de ventilación basado en tecnología naval.

Tal y como se dijo anteriormente, este sistema está sectorizado por tramos y se conducen las posibles filtraciones con colectores hasta pozos de control y registro donde son aforados.

El montaje se hizo desde coronación con unos andamios colgados que podía circular de arriba hacia abajo y sobre coronación.

9 INTERFASE CIMIENTO-LÁMINA IMPERMEABILIZANTE

La impermeabilidad del paramento de aguas arriba de la presa quedó garantizada con la lámina prevista siempre que no existan grietas en la lámina y siempre que no se produzcan filtraciones por el contorno de la misma. A su vez, la ausencia de grietas en la propia lámina quedó garantizada por el control de calidad, realizado previamente a su colocación, y por la correcta ejecución, comprobada, de todas las uniones o soldaduras entre las diferentes bandas in situ.

Más delicado fue el problema de evitar que el agua se introdujese por el contorno exterior de la lámina. El procedimiento experimentado que garantiza la impermeabilidad es el empotramiento final de la lámina, en todo su contorno inferior, en un cuerpo de hormigón en masa o ligeramente

armado, denominado “plinto” en la tecnología de presas.

El cimiento sobre el que apoya el plinto, está formado en nuestro caso por materiales paleozoicos (cuarcitas, pizarras, etc.), cruzados por numerosas litoclasas y familias de discontinuidades, como es frecuente en este tipo de rocas. Una vez lleno el embalse podrían producir, con alta probabilidad, filtraciones a través del sistema de discontinuidades de la roca contorneando inferiormente el plinto y alimentando el espacio entre la lámina impermeable y el paramento de la presa. En este caso el embalse filtraría a través del cuerpo de la presa, como si no existiera la impermeabilización realizada, a pesar del gasto realizado con pantalla y plinto.

Para evitar esta circunstancia se hizo necesario enlazar el plinto con la pantalla de impermeabilización del terreno explicada anteriormente, suficientemente profunda y densa, para eliminar cualquier filtración a través del sistema de litoclasas a pie de paramento.

El plinto receptor de la lámina y origen de la pantalla de impermeabilización del cimiento tiene un mínimo de 1,00 m para evitar su rotura por flexión y no supera (salvo excepción) la altura de 1,50 m para evitar un excesivo e innecesario volumen de hormigón. Su anchura en sentido horizontal y perpendicular al eje de presa permitió la realización de los taladros y del proceso de inyección del cimiento. Se realizaron juntas verticales en el hormigón del plinto, cada 12–15 m como máximo. Se impermeabilizó superficialmente estas juntas mediante la aplicación externa de un mastic “ad hoc”.

Por último y para dejar la presa totalmente impermeabilizada se inyectaron las juntas de retracción del plinto con resinas epoxy de alta viscosidad, mayores incluso a 10.000 CPs.

Figura 13. Hormigonado del plinto.

Figura 14. Inyección de las juntas del plinto.

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Nueva metodología para el estudio y la rehabilitación de la red de drenaje. Aplicación a la presa de Siurana (Tarragona)

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ABSTRACT: In the course of time, drainage systems of dams lose their efficiency, as a result, it is necessary to rehabilitate them. It doesn't exist a general method in order to attack this type of works and fix the main parameters. After many rehabilitations carried out among the years, HCC has developed a general method that allows to face up these works optimizing results and costs. This method has been applied in Siurana dam, located in Tarragona (Spain), with great results.

1 INTRODUCCIÓN

La red de drenaje, al cabo del tiempo, comienza a perder efectividad, debido tanto a la acumulación de material de arrastre como a la formación de concreciones calcáreas, lo que puede llevar incluso a una completa obturación de la red. Aunque cada presa tiene sus propias características de evolución, de media, se puede considerar razonable efectuar una rehabilitación de la red de drenaje cada 15 años, momento de su fase de explotación en que puede haber perdido el 50% de su capacidad inicial.

Por todo ello, y con el fin último de que la subpresión se mantenga dentro de valores admisibles, tras más de 37 años de funcionamiento, el explotador de la presa de Siurana, la Agencia Catalana del Agua, decidió convocar un concurso para la rehabilitación de la red de drenaje de cimiento de la presa, habiendo previsto en fase de proyecto la rehabilitación de la totalidad de los drenes existentes, 2.210 m.

En los siguientes apartados, se describe brevemente la presa de Siurana, se comentan algunas consideraciones generales sobre la subpresión, se explica el Método Racional de Rehabilitación de Redes de Drenaje y se incluyen las conclusiones de los trabajos realizados.

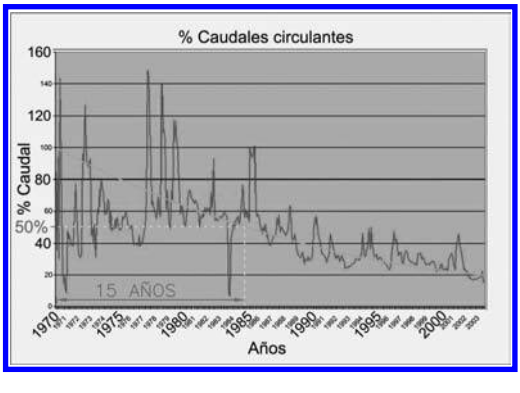

Figura 1. Histórico de aforos y ajuste de una curva logarítmica (datos de una presa, a modo de ejemplo, que no corresponde con Siurana).

2 DESCRIPCIÓN GENERAL DE LA PRESA DE SIURANA

La presa de Siurana, cuya construcción concluyó en 1.972, está situada en el cauce del río Siurana

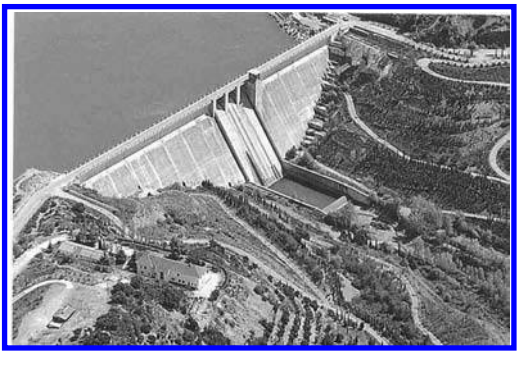

Figura 2. Foto general de la presa desde aguas abajo.

(Tarragona). Es de gravedad y planta recta. La altura total desde cimientos es 63 m. y la longitud de coronación 274 m. Los taludes son 0,05 aguas arriba y 0,73 aguas abajo.

La presa de Siurana contaba, antes de las obras que aquí se describen, con una red de drenaje de cimiento *convencional*, formada por drenes verticales de diámetro 76 mm perforados en las galerías horizontales de inspección a una equidistancia de 4 m y que penetran en roca hasta un tercio de la máxima carga de agua para cada sección.

3 ALGUNAS CONSIDERACIONES SOBRE LA ESTABILIDAD AL DESLIZAMIENTO

En el cálculo de estabilidad al deslizamiento de las presas de gravedad existen tres fuerzas activas principales: peso, empuje hidrostático y subpresión (sin tener en cuenta otras fuerzas accidentales como el sismo que tienen poca incidencia en la mayoría de presas españolas).

Las fuerzas reactivas, rozamiento y cohesión, son las que se oponen al deslizamiento de la presa y actúan en la medida que se necesitan.

De las fuerzas activas, el peso y empuje hidrostático, son perfectamente conocidas y, para una misma altura de presa, su valor es muy similar en todas ellas y no varía con el tiempo.

La subpresión, sin embargo, es la gran desconocida y su valor real varía enormemente de unas presas a otras dependiendo de factores muy diversos como: permeabilidad y diaclasado del cimiento, permeabilidad del hormigón, calidad de ejecución de la unión presa-cimiento, posibilidad de aparición de una grieta en la zona de aguas arriba, etc. Además, con el paso del tiempo su valor puede variar en función de la colmatación o lavado de diaclasas, pérdida de eficacia del drenaje, etc.

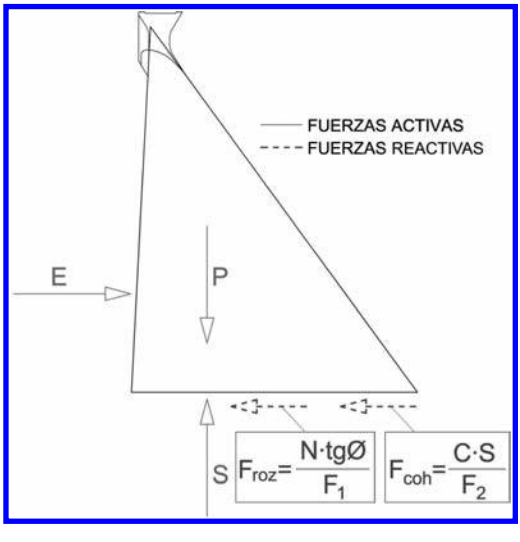

Figura 3. Fuerzas en el cálculo de estabilidad al deslizamiento.

Debido al gran desconocimiento que teníamos del valor de la subpresión (no hay definidas unas pautas de cómo ni dónde medirla), tradicionalmente se han considerado dos posibles valores de cálculo: diagrama triangular para la hipótesis de drenaje ineficaz y diagrama quebrado, con reducción a un tercio de la carga hidrostática en la vertical asociada al plano de la red de drenaje, para hipótesis de drenaje eficaz (ver Fig. 4). Con estos dos valores se han calculado y se siguen calculando la inmensa mayoría de presas de gravedad que se proyectan.

En la actualidad, pese a que el conocimiento del valor real de la subpresión sigue teniendo ciertas limitaciones (sólo lo podemos medir en puntos concretos), las incertidumbres se han reducido de forma importante. Es por ello que, dada la importancia de esta solicitación inestabilizante, consideramos fundamental que todas las presas de gravedad cuenten con una red de piezometría dedicada exclusiva-mente a la medida de subpresión en aquellos lugares donde sea de interés por motivos de seguridad, ya sea en el contacto o en otros puntos.

4 MÉTODO RACIONAL DE REHABILITACIÓN DE REDES DE DRENAJE

Después de muchas obras ejecutadas de rehabilitación de redes de drenaje y con la experiencia acumulada en ellas, HCC ha desarrollado una metodología nueva que se basa en:

- Análisis geométrico
- Análisis de datos existentes de subpresión
- Instalación de red completa de piezometría
- Análisis completo de subpresión
- Selección de zonas a tratar y método óptimo
- Rehabilitación de red de drenaje

En los siguientes apartados se describe el método aplicado a la presa de Siurana.

4.1 *Análisis geométrico*

El primer paso, antes incluso de instalar una red de piezometría que nos permita conocer la subpresión en determinados puntos de la presa, consiste en

analizar la geometría de la presa y las limitaciones de la red de drenaje existente para poder establecer las soluciones óptimas para drenar el contacto.

Un concepto, de extrema sencillez, pero del que no existe referencia alguna en la bibliografía especializada, y que no se suele tener en cuenta a la hora de dimensionar las redes de drenaje, es que los drenes verticales tratan de aliviar subpresiones en el cimiento y contacto pero desaguan en puntos a mayor cota que la correspondiente a aquellos puntos del cimiento donde se pretende aliviar la presión. Por tanto, existe una limitación geométrica que condiciona la eficacia de una red de drenaje vertical.

Por ello, en los bloques de la presa en que la distancia entre la galería y el cimiento ($H_{cont\text{-}gal}$) es elevada, como ocurre en el bloque N° 3 de la presa de Siurana (Fig. 4), la red de drenaje vertical supone conseguir una eficacia muy limitada.

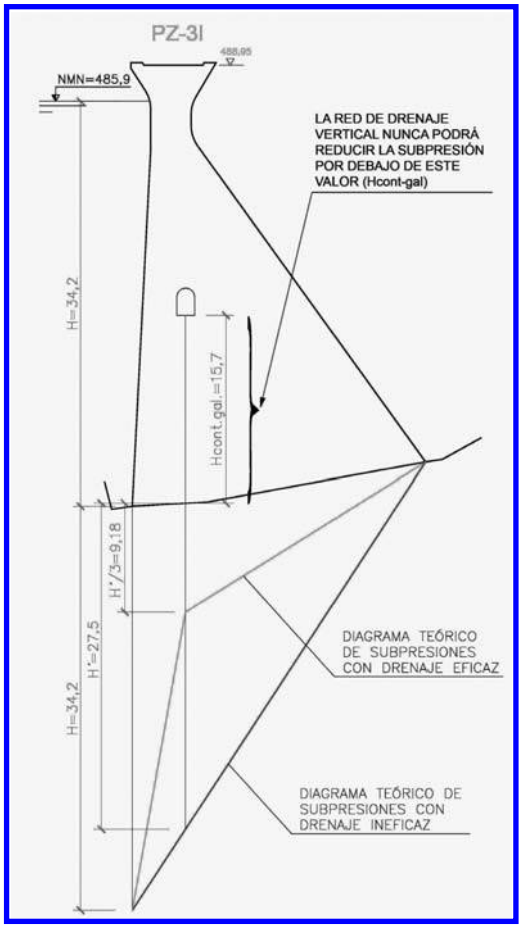

Figura 4. Sección transversal de la presa en bloque N° 3 (PZ-3I).

Como se aprecia en la tabla 1, los drenes no actúan hasta que se alcanzan valores de subpresión un 70% superiores a los teóricamente admisibles. Una prueba de la ineficacia de los drenes verticales del bloque N° 3 de la presa es que, en fase de obras y a nivel cercano a sobrevertido por compuertas de aliviadero, ninguno de ellos desagua caudal. Es decir, la subpresión existente no puede aliviarse por drenes verticales al ser inferior a la diferencia de cotas entre cimiento y cabeza de dren en galería.

Para poder considerar convenientemente esta limitación geométrica y localizar las zonas más críticas a este respecto se realiza un gráfico de subpresio-nes (ver Fig. 5), sobre el perfil longitudinal de la presa, en el que la línea de color negro representa la distancia entre la galería y el cimiento ($H_{cont\text{-}gal}$) que indica el valor mínimo de subpresión mínima para que los drenes verticales empiecen a funcionar.

4.2 *Análisis de subpresión previo*

Nunca se debe perder la perspectiva de que el objetivo de una red de drenaje es aliviar las presiones intersticiales en el cimiento, contacto (subpresión) y cuerpo de presa. Por ello, a la hora de establecer el tipo de drenaje necesario es fundamental tomar medidas de subpresiones reales y analizarlas convenientemente.

Normalmente, en la mayoría de las presas de gravedad de cierta antigüedad, no existen suficientes datos de subpresiones para realizar una diagnosis y tener un criterio adecuado para proyectar la rehabilitación del drenaje.

El caso de la presa de Siurana, únicamente se contaba con los datos de cota de agua medida en los drenes existentes antes de proceder a su rehabilitación (ver Fig. 5).

Se puede observar que las limitaciones geométricas comentadas quedan patentes y coinciden con las zonas donde la eficacia del drenaje es menor. En los bloques N° 3 y 12 los drenes no llegan a disipar subpresiones, (bien sea por grado de colmatación o por la limitación geométrica comentada anteriormente).

Tabla 1. Subpresión en bloque N° 3 con cota de agua en el N.M.N.

	H	H′	H′/3	$H_{cont\text{-}gal}$	$H_{cont\text{-}gal}$/(H′/3)
PERFIL	m.c.a	m.c.a	m.c.a	m.c.a	%
PZ-3I	34,2	27,5	9,2	15,7	171,2
PZ-3D	38,2	32,3	10,8	18,4	170,8

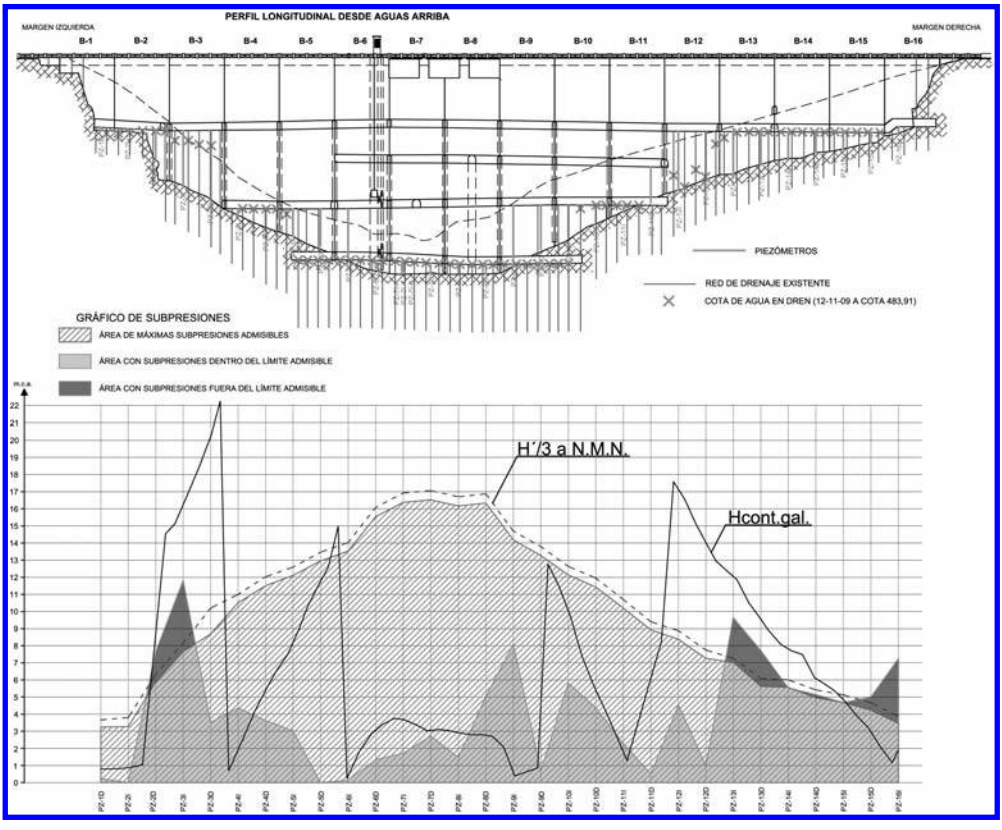

Figura 5. Perfil longitudinal con gráfico de subpresiones a 26-nov-2009 (cota agua: 483,93 m—Altura de agua respecto a cimientos en B-7: 60,77 m—97%).

4.3 *Instalación completa de red de piezometría*

Para completar la información anterior se decidió instalar una red completa de piezometría. Esta red permite obtener información muy valiosa para poder conocer la situación de la presa frente a la subpresión y su estabilidad al deslizamiento. Además, el hecho de realizarla antes de la rehabilitación del drenaje permitirá comprobar la eficacia del mismo en cada una de las fases de ejecución de nuevos drenes o rehabilitación de los existentes.

Se instalaron dos sensores de cuerda vibrante, convenientemente sellados, en cada bloque de la presa. Se ubicaron en el contacto presa-cimiento ligeramente aguas abajo de la red de drenaje (ver Fig. 5).

4.4 *Análisis completo de subpresión*

Una vez obtenidos los datos de subpresión se puede proceder al diseño de la red de drenaje óptima y, mediante la toma de medidas de los sensores instalados, a la comprobación de la eficacia conseguida en cada una de las fases de trabajo. Obviamente los datos de subpresión medidos permitirán sacar más y mejores conclusiones cuanto más cercano al N.M.N. se encuentre la cota de agua en el embalse. En el caso de la presa de Siurana el embalse estaba prácticamente lleno durante la ejecución de los trabajos por lo que los datos eran óptimos para analizar la eficacia de las actuaciones.

Las subpresiones medidas eran en general muy bajas, exceptuando zonas localizadas próximas a los estribos. Gracias a la información que se obtuvo durante la ejecución de las obras se pudo dar una explicación razonable a este hecho basado en los siguientes factores:

- La unión entre el cimiento de la presa y el hormigón de la misma es excelente. Esto tiene gran importancia porque este contacto es, normalmente, una de las principales vías de filtración y de transmisión de subpresión en las presas.
- El cimiento, formado principalmente por pizarras finas negras, es muy impermeable. Parte de

Figura 6. Contacto monolítico hormigón-cimiento en un testigo de Ø46 mm.

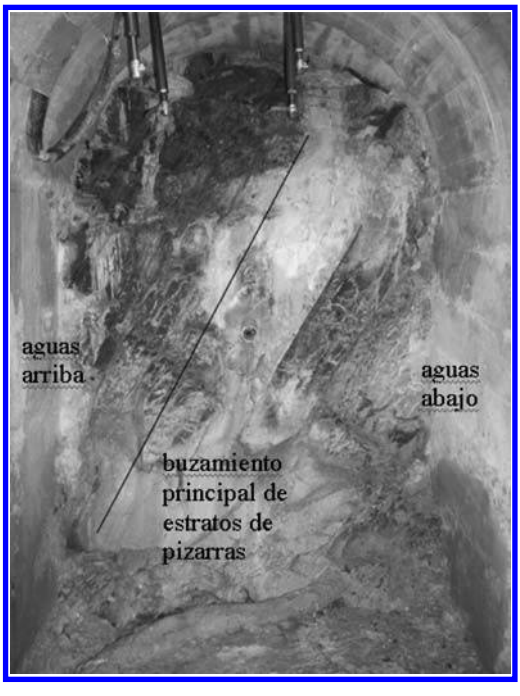

Figura 7. Buzamiento principal de los estratos de pizarras en encuentro de la galería horizontal con estribo izquierdo.

esta impermeabilidad se debe a una orientación favorable de la dirección de la esquistosidad de las pizarras. Esto también concuerda con los datos de aforos de la red de drenaje que arrojaban caudales filtrados muy bajos.

4.5 *Selección de zonas a tratar y método óptimo*

Tras el análisis de todos los datos recabados se propuso la solución más adecuada para el refuerzo de la red de drenaje que consistió en:

– Perforación de nuevos drenes en abanico desde la galería horizontal N° 3 en ambos estribos. Estos trabajos permitirán drenar aquellas zonas en que existía limitación geométrica entre boca de dren y boca de taladro.
– Perforación de nuevos drenes verticales en el bloque N° 16 para reforzar dicha zona, donde se midieron subpresiones superiores a las admisibles.
– Rehabilitación de zonas concretas del drenaje existente mediante la tecnología de agua a alta presión y elevado caudal.

Todas estas actuaciones permitieron reducir las subpresiones hasta valores inferiores a los máximos teóricos admisibles.

4.6 *Rehabilitación de la red de drenaje*

Tal y como se aprecia en la Fig. 9, mediante la ejecución de un número reducido de nuevos drenes y la rehabilitación de algunos drenes con agua a presión, en zonas concretas, se han conseguido unos resultados muy satisfactorios. Además no se ha actuado en toda la zona central de la presa ya que no es necesario puesto que las subpresiones eran muy bajas. Los valores de subpresión finales están muy por debajo de los valores teóricos para

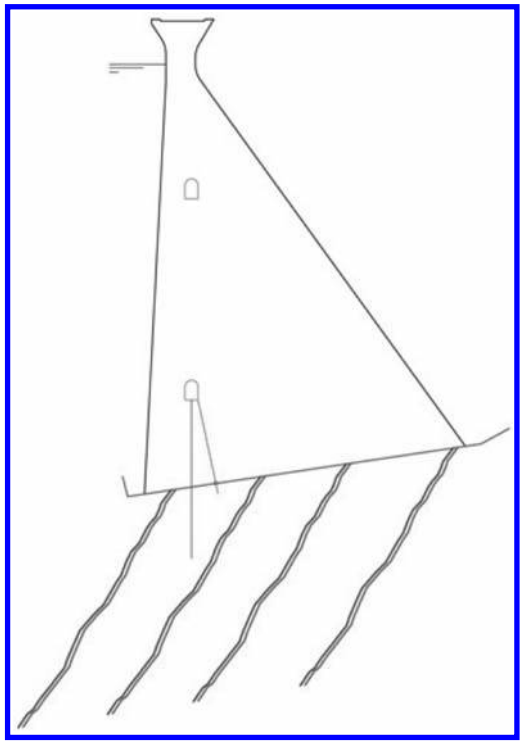

Figura 8. Croquis buzamiento principal de los estratos de pizarras.

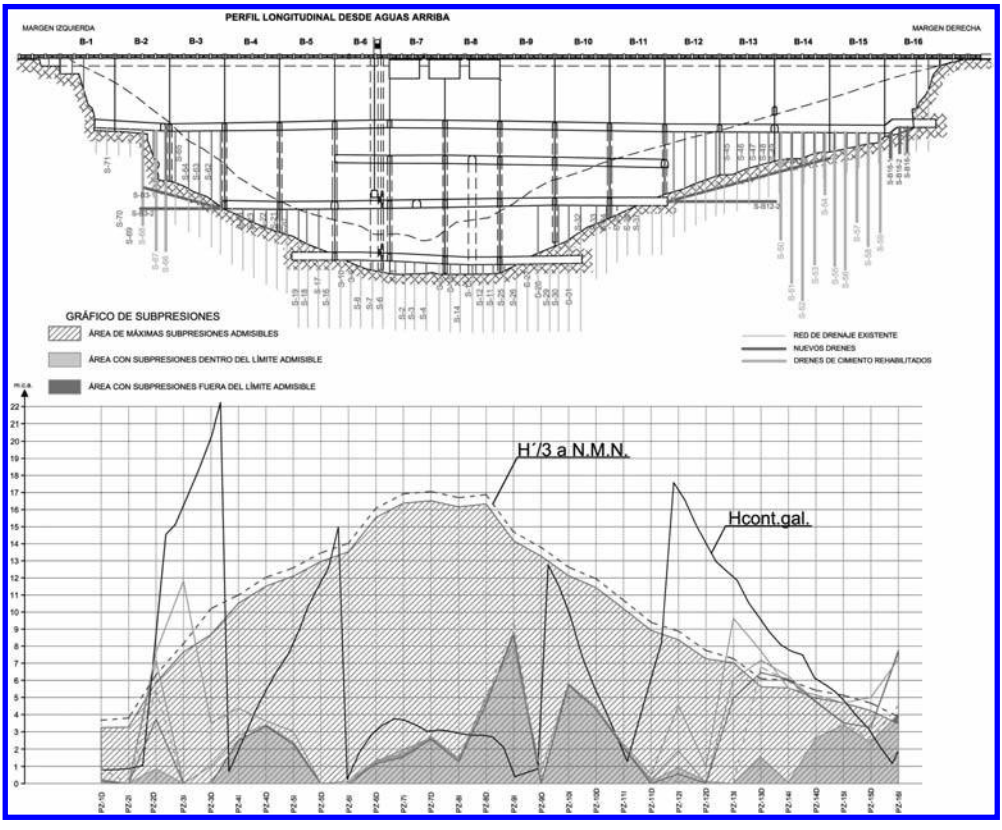

Figura 9. Perfil longitudinal con gráfico de subpresiones a 22-ene-2010 (cota agua: 484,64 m—Altura de agua respecto a cimientos en B-7: 61,48 m—98%).

drenaje eficaz lo que garantiza unos coeficientes de seguridad mayores a los exigidos por la instrucción en la estabilidad al deslizamiento.

Para tener una idea de la importancia de la subpresión así como la reducción conseguida, se ha calculado este valor en toda la superficie de contacto del bloque 3. La subpresión ha pasado de 3.260 tn, antes del refuerzo del drenaje, a 1.640 tn después de la obra. La reducción conseguida es de 1.620 tn (50%). Aunque este valor hay que tomarlo con cierta cautela nos podemos hacer una idea de lo mucho que podemos mejorar la estabilidad al deslizamiento reduciendo la subpresión en las presas de gravedad.

5 CONCLUSIONES

La subpresión es una fuerza desfavorable y de una magnitud muy importante en el cálculo de estabilidad al deslizamiento de presas de gravedad. Esta fuerza varía enormemente de unas presas a otras e incluso en una misma presa, a lo largo del tiempo.

Debido al desconocimiento del valor real de la subpresión se han adoptado unos modelos de cálculo que, en ocasiones, se alejan mucho de la realidad.

Las redes de drenaje son el principal instrumento para la reducción de la subpresión pero en la mayoría de los casos se diseñan y construyen según unas reglas teóricas que en muchas ocasiones no son eficaces.

La red de drenaje de una presa tiene que adaptarse a las condiciones reales de dicha presa (geometría, cimiento, etc), que son únicas. Por tanto hay que huir de soluciones teóricas generales y estudiar cada caso concreto para diseñar la red de drenaje más eficaz posible.

Mediante la metodología aquí expuesta, se consigue un diseño a medida de la red de drenaje óptima para cada presa. Instalar una red de

piezometría completa resulta fundamental, tanto para el diseño de la red de drenaje como para la comprobación de la eficacia de los trabajos ejecutados y el seguimiento y control futuro de las subpresiones existentes. Los ahorros económicos que se consiguen son muy variables dependiendo de cada caso concreto pero pueden llegar a valores muy altos de hasta el 80% del conjunto de la obra.

Para poder poner en práctica la metodología aquí descrita, es necesario contar con: personal técnico cualificado, capaz de analizar los datos y tomar decisiones rápidas, instrumentos de investigación que aporten información necesaria como son las cámaras de TV, que permiten la inspección de taladros, y los ensayos geofísicos con sondas que permiten detectar y cuantificar las principales vías de filtración a lo largo de toda longitud operativa de un dren.

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Nuevos materiales y tecnologías para el sellado de filtraciones en hormigón y roca sin afectar a la explotación. Resinas visco expansivas

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ABSTRACT: One of the problems that often appears in the repair works of dams, is the need to emptying the reservoir. New materials and technologies developed in Spain in the last decade avoid this problem, allowing the sealing of important leakages and the recovery of the structural monolithism, even under high pressure water, without polluting the environment. The use of high viscosity and thixotropy polymers, almost viscoelastic solids, in addition to the desing of new and powerful pumps, able to inject this polymers up to 600 atm, have enabled important work in more than 100 dams, repaired without affecting the explotation.

1 ANTECEDENTES

La solución de determinados problemas en presas, como el sellado de fisuras, la estanquidad de juntas, el despegue del tacón de aguas arriba en las de gravedad, el lavado del núcleo de arcilla en las de materiales sueltos o el paso de importantes caudales por litoclasas profundas, requerían generalmente el vaciado previo del embalse, antes de acometer la reparación.

Las reparaciones tradicionales, basadas en la inyección con cemento, requieren que no exista circulación de agua, para evitar los arrastres de la lechada. El desembalse tiene unos costes económicos, generalmente importantes, a los que se une el lucro cesante, el coste medioambiental, y muchas veces el coste político para la autoridad que lo autoriza. En general el vaciado supone una pérdida económica importante para la propiedad o una merma de garantía en el servicio, especialmente grave para las presas adscritas al abastecimiento.

Si se consigue desembalsar, la reparación se realiza a ciegas y sólo es factible comprobar su resultado, una vez que se ha vuelto a llenar el embalse. Es relativamente frecuente que en trabajos de este tipo, una vez alcanzada la cota de explotación, se aprecie que las filtraciones, o el problema original ha disminuido, pero no se han conseguido solucionar por completo.

La formulación de polímeros especialmente diseñados para la ejecución de estos trabajos, ha permitido, en los últimos años, la solución de este tipo de problemas, con embalse lleno, sin afectar a la explotación. Trabajar con cota máxima de agua, obvia todos los inconvenientes de las soluciones tradicionales. Efectivamente, ya no se trabaja a ciegas, por lo que una vez solventado el problema, ya no volverá a aparecer.

Desde el punto de vista económico, aunque este tipo de material es notablemente más caro que el cemento, como no se produce lucro cesante, al mantener la presa y sus instalaciones en servicio, el importe final de la operación, suele ser netamente favorable a esta nueva tecnología. Ello sin

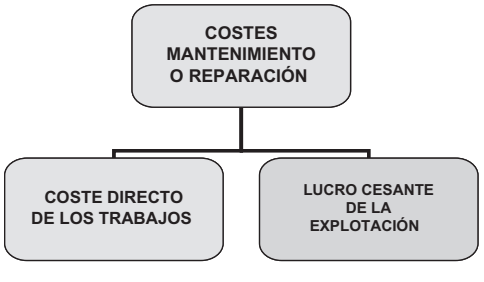

Figura 1. Costes totales de mantenimiento o reparación.

contar con la mayor eficacia de estos productos y por ello su menor consumo para iguales logros.

2 MATERIALES

2.1 *Formulaciones epoxídicas*

Las resinas epoxi, son polímeros termoestables que endurecen al mezclarlos con su endurecedor o catalizador correspondiente. Comenzaron a ser comercializados, inicialmente como adhesivos, en el primer tercio del siglo XX. Además de su faceta de potente adhesivo, se empezaron a utilizar en aplicaciones superficiales, como capas de protección frente a la corrosión o imprimaciones para mejorar la adherencia entre un sustrato y la pintura final. El interior de los botes de conservas ácidas, o los tratamientos anticorrosión de los coches, suelen ser también epoxis.

Ya en los años 50, se empezaron a utilizar para la ejecución de pavimentos industriales, el encapsulamiento de material eléctrico, como circuitos impresos, o, para la fabricación de laminados, de los que el máximo exponente actual son las palas de los aerogeneradores que siembran nuestro paisaje.La industria de las resinas epoxi mueve anualmente más de 10.000 millones de € en el mundo, utilizando prácticamente todo el material en las aplicaciones anteriormente mencionadas. Resulta significativo, que en enciclopedias, como la muy extendida Wilkipedia, no mencione entre las aplicaciones de las resinas epoxi su utilización para la resolución de problemas en ingeniería civil.

Se han venido utilizando, para nuestras aplicaciones, epoxis que estaban destinados a otros fines. Una gran parte de las resinas comerciales tienen una resistencia a compresión cercana a 1.000 kp/cm^2 y de más de 400 kp/cm^2 a tracción. Es evidente que inyectar un hormigón con un material de esas características es un auténtico derroche económico que no tiene sentido alguno.

El otro inconveniente de las resinas epoxi comerciales es su baja viscosidad, lo que no las hace aptas para la inyección en fuertes corrientes de agua, ya que son fácilmente arrastrables. El análisis de las fichas técnicas de los principales fabricantes de estos materiales, nos da valores de la viscosidad, generalmente entre 400 cP y 10.000 cP, (recordemos que la del agua es 1cP), sin embargo para lograr sellar las filtraciones que se pueden presentar en una presa, se requieren valores superiores a 100.000 cP.

Efectivamente, el tratamiento de filtraciones de varios centenares de litros por segundo, requiere que el material que se inyecte no sea ni miscible ni arrastrable por el agua, y eso sólo se consigue con masillas de muy alta viscosidad, y estos polímeros no se encuentran en los catálogos de los fabricantes.

Además de alta viscosidad, hemos comprobado que es imprescindible que el polímero tenga una elevada tixotropía. Recordaremos que la tixotropía es una característica de los fluidos no newtonianos, que pierden viscosidad cuando se les somete a una tensión cortante, por ejemplo al batirlos o al desplazarse por una fisura cuando son inyectados en ella. Gracias a la tixotropía, el material avanza por las fisuras mientras se inyectan asimilando su comportamiento al de un líquido, pero se convierte en una masilla muy viscosa en cuanto se para la bomba. Esta propiedad es esencial para el control del avance del material, ya que no es arrastrable por el agua y sólo se extiende por la fisura cuando es empujado por la presión de la bomba.

Conseguir el avance de este tipo de masillas, primero a través de las perforaciones y luego, dentro de las fisuras, de espesores a veces submilimétricos, requiere un importante aporte de energía al material, o lo que es lo mismo, que la bomba sea capaz de lograr altas presiones. Las máquinas que empleamos en estos momentos son capaces de alcanzar 600 bar a la salida de bomba. Por lo tanto este tipo de inyección puede ser considerada como activa, ya que la resina desplaza el agua y se controla con bastante precisión su circulación por las fisuras.

Evidentemente, a propiedades anteriores hay que añadir que el polímero endurezca y adhiera bajo agua y que no contamine. Con estos materiales, no solamente se consigue sellar importantes

Figura 2. Ejemplo de masilla de alta visco tixotropía.

vías de agua, hasta de centenares de litros por segundo, sino que, al adherir y tener elevadas características mecánicas, devuelve el monolitismo perdido a la estructura objeto de tratamiento.

2.2 *Resinas acuarreactivas*

Son resinas de poliuretano que tienen la propiedad de espumear en presencia de agua, reteniendo el gas que se forma dentro del propio polímero que va endureciendo al tiempo. Se consigue una estructura de poros cerrados, que logra volúmenes que pueden alcanzar más de veinte veces el inicial de resina.

El grado de espumeo, y por lo tanto las características finales del material dependen del grado de mezcla con el agua que haya tenido a su disposición. Dado que esa mezcla se produce de forma natural al inyectar la resina dentro de una zona húmeda o en una corriente de agua, el aplicador no puede controlar ni dónde se va a producir el hinchamiento, ni cuánto va a espumear el material. Es por lo tanto una técnica pasiva, ya que inyectamos una resina en una corriente de agua, que al arrastrarla efectúa su mezclado, sin que podamos conocer a priori el efecto que va a producir. Al ser muy fluidas, generalmente por debajo de los 1.000 cP, son arrastradas por la corriente, sin que tengamos control alguno sobre la zona de espumeo.

Evidentemente este material no consigue devolver el monolitismo a la estructura y es tan solo de aplicación, siempre con precauciones, para eliminar pequeñas filtraciones o humedades. En el caso de tratamiento de juntas, tampoco es muy recomendable. Efectivamente, una vez polimerizada, la espuma es un material aparentemente elástico. Su elasticidad se debe a la inclusión de burbujas de gas en la resina endurecida lo que hemos comprobado, que, sometida a importantes compresiones, como las que tendrían lugar al cerrarse una junta, estas oclusiones implosionan, no recuperando, al volver a contraerse la estructura, su volumen inicial y dejando, por lo tanto, vías para el paso del agua.

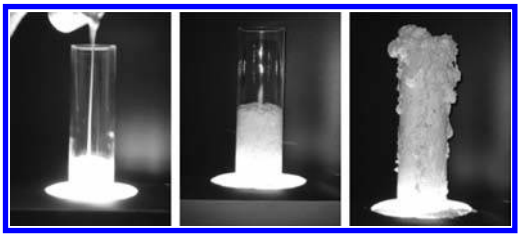

Figura 3. Secuencia del aumento de volumen de una resina acuarreactiva.

Su puesta en obra se realiza con medios convencionales, esto es con bombas comerciales aptas para líquidos, que trabajan presiones generalmente inferiores a 60 atm. Este tipo de material es muy popular, debido a su facilidad de aplicación, puede resultar adecuado para elimininar humedades en sótanos, pero no para sellar filtraciones en presas.

2.3 *Resinas visco expansivas*

En los dos últimos años hemos tratado de desarrollar un nuevo material que tuviera las ventajas de los epoxi como visco tixotropía, no ser arrastrables por el agua, no contaminar, adherir al hormigón y a la roca, tener altas características mecánicas, y la que más nos interesaba de las acuarreactivas: su capacidad de expandir. En este caso queríamos que se produjera independientemente de la presencia o no de agua.

Esta propiedad de hinchamiento es muy importante cuando se trata de rellenar no fisuras regulares, sino hormigones porosos, contacto de la cimentación o litoclasas de la roca, en definitiva, donde existan importantes oquedades. Efectivamente inyectar con resinas epoxi huecos de cierta entidad, puede ser económicamente inviable, sin embargo realizarlo con un material que, independientemente de que haya o no agua, pueda hinchar treinta veces su volumen inicial, disminuye, nada menos que por treinta los costes del trabajo.

A finales del 2009, se realizó la primera inyección de prueba con este material, en la Presa de La Minilla, (Sevilla, Emasesa), y en enero de 2010 se comenzó ya a gran escala en la presa de La Ribeira (La Coruña, Endesa). Los resultados, en ambas, estuvieron muy por encima de las expectativas, como se comentará posteriormente.

Figura 4. Testigos extraídos del hormigón mal vibrado de una presa, con coqueras generalizadas. El testigo superior ha sido inyectado con una resina que rellena los huecos.

3 BOMBA DE INYECCIÓN

Para efectuar la inyección de este nuevo material cotábamos ya con bombas de diseño propio, capaces de bombear masillas de elevadas viscosidades, más de 150.000 cP, a largas distancias, pudiendo presurizar las resinas hasta a 600 atm. Pero teníamos un serio inconveniente aún si resolver, como es el batido de las resinas. Efectivamente, todos estos polímeros vienen en dos componentes, lo que requiere su mezcla antes de la inyección, operación que se viene realizando habitualmente de forma manual.

El batido manual tiene numerosos inconvenientes.

La cantidad mezclada ha de ser inyectada rápidamente ya que tiene un tiempo de comienzo de la reacción rápido. Ello obliga, si el proceso de inyección no avanza a la velocidad prevista, a tener que desperdiciar el material mezclado.

Un batido escaso, y hay que tener en cuenta que a veces se trabaja en galerías estrechas, con urgencia por atajar alguna vía de agua importante, y por lo tanto en condiciones de estrés para los operarios, lleva a un grado de polimerización escaso e irregular. Esto puede conducir a que el material inyectado no llegue a polimerizar nunca, o sus características finales sean manifiestamente inadecuadas.

Por el contrario un exceso de celo en el batido, puede ser incluso más perjudicial. Efectivamente, demasiado tiempo de agitación se ha comprobado que tiende a incluir burbujas de aire en la masa, pero sobre todo, produce un indeseable calentamiento de la resina. Todos estos polímeros, por un lado, tienen una reacción de polimerización fuertemente exotérmica, lo que en una fisura no tiene importancia, ya que la superficie del material inyectado, en contacto con el hormigón o la roca es muy importante y, por lo tanto, el entorno absorbe el aumento de calor. Sin embargo, en un bote, durante el batido, o en la tolva de abastecimiento del material a la bomba, no hay posibilidad alguna de absorción del calor producido.

El problema estriba en que estos materiales, siguen la ley de Arrhenius, según la cual al aumentar en 10°C la temperatura de una reacción química, su velocidad se duplica. Esto hace que si el operario se descuida y agita algunos minutos de más la mezcla, la resina puede llegar a una combustión espontánea en el bote, o endurecer, de forma anticipada durante la inyección, acarreando, en ambos casos, importantes problemas.

Con objeto de evitar lo anterior, hemos diseñado un nuevo tipo de inyectadora que realiza la mezcla de forma automática, en tan sólo unos segundos.

Con esta nueva máquina, cada componente de la resina se vierte en una tolva independiente,

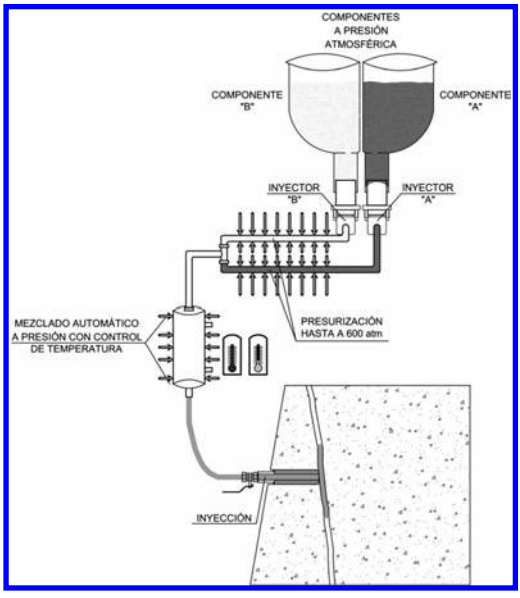

Figura 5. Esquema inyectadora.

de donde son presurizados hasta 600 atm, de allí pasan al agitador, que logra una mezcla perfecta y homogénea. Además este agitador tiene la posibilidad de adecuar térmicamente el material antes de proceder a su inyección.

Esta bomba es apta tanto para los epoxis tradicionales como para los nuevos materiales visco expansivos.

4 APLICACIONES

Esta tecnología que permite la inyección de polímeros extraordinariamente visco tixotrópicos, tiene su principal aplicación en la reparación de presas, en el sellado de filtraciones sin afectar a la explotación, esto es sin necesidad de desembalsar. De esta forma se han podido, en los últimos años, sellar filtraciones de 900 lts/sg, o inyectar y reparar bajo más de 100 m de columna de agua.

También se han inyectado juntas verticales, tanto en presas bóveda como de gravedad, siempre sin necesidad de desembalsar.

La introducción de las nuevas resinas visco expansivas, ha permitido solucionar, con costes asumibles, problemas, hasta ahora inabordables. Efectivamente, el sellado del tacón de aguas arriba de presas de gravedad, cuando, a embalse lleno, se produce un despegue con el consiguiente y peligroso aumento de subpresiones, sólo es solucionable con estos materiales y tecnología.

De igual modo, el sellado de litoclasas en la roca de cimentación, o el consolidado del contacto,

cuanto se encuentra muy deteriorado, eran problemas, que no se podían abordar, hasta ahora con embalse lleno.

La gran ventaja de trabajar con máxima cota de agua es que no se interrumpe la explotación, evitando el lucro cesante que un hipotético desembalse produciría. Además sellar o reparar en las peores condiciones posibles, es una garantía de que la reparación va a ser duradera.

A continuación se describen dos ejemplos de trabajos de reparación realizados con estos polímeros visco expansivos.

5 PRESA DE LA MINILLA

Los problemas de la presa de La Minilla, situada en Sevilla y cuyo titular es EMASESA, se han descrito en una ponencia aportada a este mismo congreso, firmada por D. Fernando Vázquez y D. Iñaki Escuder. En síntesis se trata de una presa de gravedad cimentada sobre piritas que, además utilizó estas mismas piritas como áridos en la confección del hormigón.

Independientemente de problemas de fisuración y filtraciones en las galerías, el hormigón del contacto con la cimentación estaba especialmente descompuesto. Cuando se trataron de tomar muestras del contacto, cara a realizar un análisis de riesgo, se comprobó que era imposible extraer testigos, ya que se desmenuzaban y no aportaban dato alguno de utilidad.

La inspección con cámara de video a través de los taladros ejecutados, mostraba que, como consecuencia de la propia perforación, se producían cavernas, lo que demostraba, como se intuía, que el hormigón y la propia roca se hallaban tan deteriorados en esta zona, que se comportaban como un material granular. Evidentemente, era nuestra propia acción, a la hora de tratar de extraer testigos, la que provocaba una importante alteración del conjunto.

A la vista de lo anterior, se decidió realizar una inyección de consolidación. La presa, que abastece de agua a Sevilla, tiene muy pocas fluctuaciones de cota de explotación y era absolutamente imposible proceder a su vaciado. Por lo tanto el material de inyección, tenía que tener las siguientes propiedades:

- No ser miscible ni arrastrable por el agua.
- No producir contaminación alguna.
- Rellenar las fisuras existentes.
- Encapsular las piritas aún activas.
- Consolidar las zonas granulares.
- Aumentar de volumen, sin necesidad de presencia de agua.

Lograr que el material que cubriera todos estos requerimientos ha costado dos años de investigación y ensayos. Finalmente a finales de diciembre de 2009, se estuvo en condiciones de iniciar los trabajos.

El proceso comienza con la realización de perforaciones de diámetro 46 mm, a rotación, espaciados, aproximadamente 1,5 m. Posteriormente se inyecta la resina visco expansiva por el primer taladro, cambiando, de forma secuencial al siguiente, a medida que la inyección avanza en la zona a tratar.

Figura 6. Testigo extraído de longitud teórica de cinco metros, del que tan sólo se consigue recuperar, escasamente un metros.

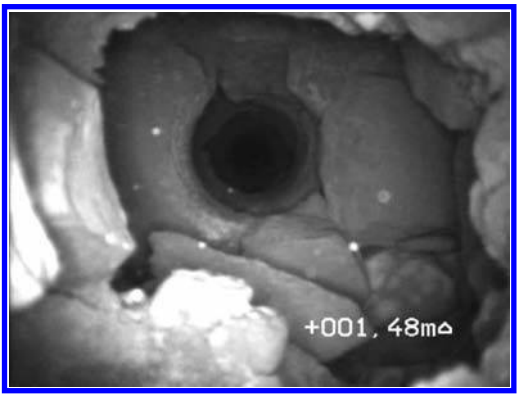

Figura 7. Fotograma de la inspección con cámara de TV de una perforación, donde se aprecian numerosas fracturas.

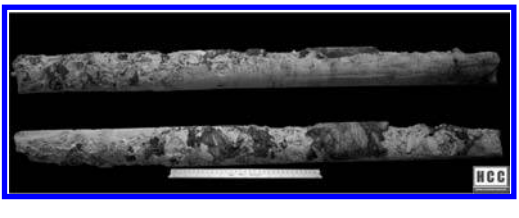

Figura 8. Imagen de un testigo extraído del contacto.

Transcurridos unos días, se extrajeron testigos de diámetro 100 mm de la zona inyectada. La primera conclusión era obvia: se conseguía obtener testigos de un metro de longitud sin fracturar. (Fig. 8). cción.

Además se pudo comprobar que no solamente la resina había monolitizado fragmentos más o menos grandes del hormigón o la roca del contacto, sino que había penetrado en las zonas más arenosas, formando un auténtico mortero.

6 PRESA DE LA RIBEIRA

La presa de La Ribeira, situada en el río Eume, en La Coruña, manifestaba importantes subpresiones, sobre todo en los estribos. Otra ponencia de D. Julián Alonso, presentada en este congreso, da cuenta detallada de los problemas encontrados y las soluciones aplicadas. No obstante hemos querido aportar, en la presente la utilidad que tuvieron las resinas visco expansivas.

La limpieza con agua a presión de los drenes, produjo un alivio de las importantes subpresiones existentes, pero se detectó que la red de drenaje era insuficiente, por lo que se empezaron a perforar nuevo drenes. Para localizar las zonas de aportación, y por lo tanto optimizar la profundidad a la que era necesario perforar los drenes, se utilizó un equipo de geofísica (Fig. 9), consistente en un ordenador, un cabrestante motorizado comandado por el ordenador, unas sondas que emiten señales eléctricas recogidas por un data logger, que las transforma y envía al ordenador.

Se utilizaron sondas de temperatura y conductividad para determinar con precisión la profundidad a la que se producían los aportes de agua al dren, y de caudal para valorar su importancia. De esta forma se pudo conocer que existían dos zonas principales de aportes, a los 10 y 25 metros de profundidad.

Con estos datos se comenzó la perforación de los drenes, monitorizando en tiempo real el resultado, gracias a una densa red de piezómetros de cuerda vibrante instalados previamente.

Se comprobó que, a medida que se perforaban los nuevos drenes, como era de esperar, se reducían las subpresiones. Pero llegados a un cierto punto (Fig. 10), las aportaciones de los drenes, con más de 200 lt/mn, aconsejaron parar estos trabajos.

Para disminuir los importantes aforos de los drenes, era obvio que se requería realizar pantallas de impermeabilización hacia aguas arriba de la presa, muy localizadas, para sellar las litoclasas por donde se producían los aportes.

Una vez más el principal condicionante era la imposibilidad de proceder al vaciado de la presa, los trabajos debían ser ejecutados con máxima cota de embalse. Esto hizo desechar lógicamente la inyección con cemento, así que se intentó con masillas epoxi. Se consiguieron resultados esperanzadores pero los elevados consumos hacían inviable económicamente la solución. El empleo de resinas acuarreactivas tampoco era la solución, ya que hubieran sido arrastradas por las fuertes corrientes existentes.

La obra estuvo parada un año, hasta que estuvieron puestas a punto las resinas visco expansivas con las que se comenzó a inyectar. Desde el primer momento se empezaron a conseguir disminuciones importantes en los aforos, y como consecuencia, al bajar sensiblemente el caudal de aportación en las litoclasas tratadas, los drenes eran capaces de trabajar adecuadamente, lográndose el objetivo final, la reducción de las subpresiones a valores admisibles.

En testigos extraídos de zonas tratadas, se aprecia (Fig. 11), el importante espesor de las litoclasas, perfectamente sellado por la resina, que ha expandido en su interior.

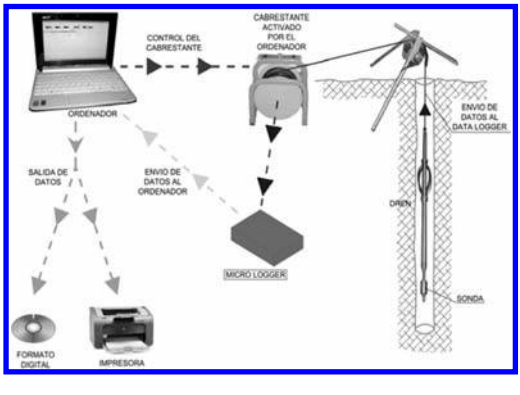

Figura 9. Esquema del equipo de geofísica.

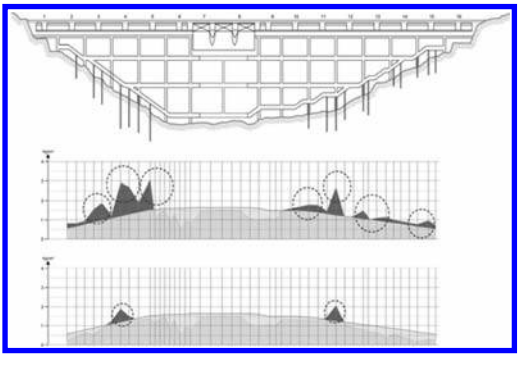

Figura 10. Subpresiones iniciales y restantes en el momento de parar los trabajos de drenaje.

Figura 11. Testigo donde se aprecia el espesor de varios milímetros de una litoclasa inyectada.

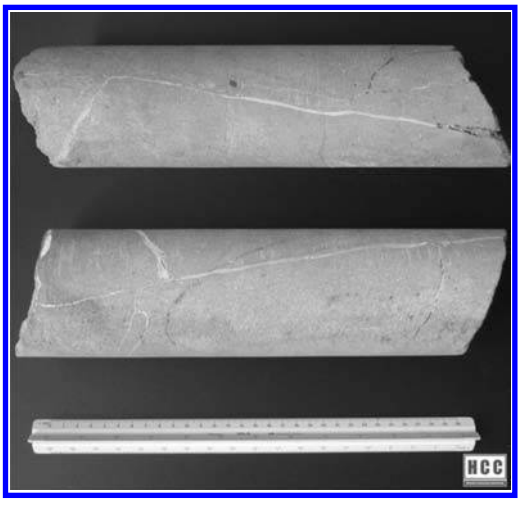

Figura 12. Testigo de roca inyectado, donde se aprecian, en blanco, zonas inyectadas con resina visco expansiva, que no sólo sella, sino que adhiere, devolviendo el monolitismo a la roca fracturada.

7 CONCLUSIONES

La inyección de polímeros visco expansivos a muy alta presión permite solucionar de manera definitiva, problemas hasta ahora inabordables. Aunque este material no requiere presencia de agua para hinchar, se ha comprobado que en ambiente húmedo, o sumergido, la expansión, aunque más moderada continúa durante días. Este fenómeno permite que el material siga rellenando, y por lo tanto mejorando la calidad del trabajo, una vez finalizada la inyección.

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La rehabilitación del aliviadero de la presa de Portodemouros

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ABSTRACT: The dam of Portodemouros, owned by Natural Gas Fenosa, is located in the province of Lugo entered service in 1968. The spillway is placed in the left bank, parallel to the dam, of fixed lip and 130 meters of length. Its capacity is 1,895 m^3/sg. The spillway has the form of a gravity dam with a horizontal gallery placed 2 meters from foundation.

In 1973, as consequence of the appearance of several cracks, the instrumentation and observation of the spillway is intensified. In the following years several interventions are carried out accompanied of a meticulous monitoring of the cracking evolution. In 1979, a structural study concludes that cracking is not originated due to the loads or to the thermal solicitations that act on the spillway. In the 80's, other study concludes, taking into consideration the monitoring of instrumentation and several tests, that the cause of cracking is an expansive phenomenon of the concrete.

In order to reduce horizontal stress provoked by the expansive phenomenon several transverse cuts of the structure are realized with great success. In addition, waterproofing and cooling tests are carried out, of doubtful results.

In 2003, Natural Gas Fenosa decides to approach an integral rehabilitation of the spillway, which would allow raising the water level of the reservoir, limited up to this moment, due to the bad situation of the structure.

Works consisted in:

- Conventional injection with cement of the interface concrete—rock.
- Cracks inyection at high pressure with high viscosity epoxi resins
- High resistance bars were placed in boreholes penetrating 2 m in foundation in order to assure the stability.

In 2010, works have been completed by the installation of 16 vibrating wire piezometers and diverse drains.

1 INTRODUCCIÓN

De todos es conocido cómo los fenómenos expansivos que se producen en el hormigón de nuestras presas influyen negativamente en la conservación estructural de las mismas. A lo largo de los años, se han detectado más y más presas afectadas por este tipo de fenómenos, obligando a sus titulares a la realización de proyectos y obras de rehabilitación estructural y mitigación de los posibles efectos negativos aparejados a este tipo de problemáticas.

El objeto principal de esta ponencia es el de dar a conocer la gran cantidad de trabajos que se han realizado en el aliviadero de la presa de Portodemouros, enfocados tanto a mitigar o hacer desaparecer el fenómeno expansivo propio del hormigón de la presa, como a rehabilitar tanto el hormigón como la cimentación.

Además, creemos interesante explicar cómo ciertas decisiones se ha gestionado el problema desde el punto de vista empresarial, y cómo, haciendo hincapié en el enfoque de un problema no demasiado conocido en sus inicios puede provocar susceptibilidades difíciles de rebatir.

2 ALIVIADERO DE LA PRESA DE PORTODEMOUROS

La presa de Portodemouros es una presa de materiales sueltos con núcleo de arcilla de 93 m de altura y 460 m de longitud de coronación.

El aliviadero lateral situado en la margen izquierda y contiguo a la coronación de la presa es de labio fijo con reducida lámina vertiente y 130 m de longitud.

Dispone de una cubeta tranquilizante de la que parte el canal de descarga sobre el que se ha construido un puente para restablecer la carretera de coronación de presa. Este aliviadero constituye realmente una presa de gravedad de unos 20 m de altura máxima que está provista de una galería situada unos 2 m sobre su cimentación.

3 CRONOLOGÍA DE LAS ACTUACIONES EN EL ALIVIADERO DE LA PRESA DE PORTODEMOUROS

1973.- Se detecta y localizan algunas fisuras en el hormigón de la galería del aliviadero.

1976.- Se instalan bases de elongámetro para medir evoluciones de fisuras.

1976.- (Diciembre) Incremento importante de caudales filtrados por drenes que aconseja un tratamiento de inyección y una nueva red de drenaje.

1977.- (Junio) Se realiza una inyección de fisuras del hormigón a base de resinas.

1979.- Se detectan nuevas fisuras en el aliviadero.

1979.- Se encarga un estudio para conocer las causas del fenómeno y posibles remedios.

1979.- Se recibe informe, recomendando ampliar sistema de auscultación, reforzar los bloques 13 y 14 (con barras de acero), hacer una junta que evite compresiones y reforzar los dos primeros bloques (con vigas).

1980.- Se realizan anclajes pasivos (sin tensar) en los bloques 13 y 14. Se inyectan fisuras con resinas.

1981.- Se montan nuevos aparatos de auscultación (extensómetros de varilla, péndulos, termopares y extensómetro vertical).

1983.- Se realizan ensayos para conocer el estado tensional del hormigón. Se indica que el embalse no supere la cota 248.

1983.- Ensayos de liberación de tensiones y se deduce compresión según el eje del aliviadero.

1983.- Se envían muestras de testigos para analizar presencia de sulfuros, pirrotitas, piritas amorfas, etc.

1983.- Hasta nueva aviso se indica que el embalse no supere la cota 248.

1984.- Se controlan las voladuras de la cantera.

1984.- Se mantiene la cota 248 como máximo embalse.

1984.- Se incrementan los dispositivos de auscultación con extensómetros de gran base, de varilla, clinómetros, medidores de convergencia y comparadores. Se realizan ensayos de probetas en laboratorio.

1984.- Se adjudica el tratamiento superficial del aliviadero.

1984.- Se impermeabilizan los bloques del aliviadero.

1984.- Informe “Aliviadero de Portodemouros. Su comportamiento hasta finales de 1983 estudiado a través de los datos de auscultación”. Causa: Expansión por ataques de sulfatos procedentes de oxidación de sulfuros existentes en los áridos. Acciones: Arriostramiento de los bloques 13, 14 y 15, tratamiento de cimentación, inyección y drenaje, apertura de dos juntas y sellado posterior e impermeabilización de paramentos exteriores.

1985.- Comienzo de inyección, micro pilotes, etc. Perforación para los péndulos.

1985.- (Diciembre) Terminación de los trabajos de tratamiento de la cimentación y estructuras de arriostramiento de los muros cajeros. Apertura de la junta transversal, impermeabilización de varios bloques e inyección de resina en fisuras de los bloques 1 al 15.

1985.- Se realizan 37 anclajes de longitud de 10 m, en los bloques 13 al 15.

1986.- Trabajos de inyección.

1986.- Finaliza el “Estudio de los resultados de exámenes y ensayos conducentes al conocimiento de las causas de expansión” del hormigón del aliviadero de la Presa de Portodemouros.

1986.- Impermeabilización realizada en los bloques del 9 al 15.

1986.- Se firma el convenio con el CEDEX para realizar un estudio de las causas de la fisuración del aliviadero.

1987.- Estudio de análisis detallado de los datos de los aparatos de auscultación.

1994.- Realización de 4 cortes transversales y su impermeabilización. Se anula la limitación de cota de embalse, pudiendo llenar hasta la cota 246 y posteriormente subir a la velocidad de 1 m por semana.

1996.- Se realiza la limpieza de drenes.

1998.- Se impermeabilizan los bloques del 10 al 13 y se instala el sistema de riego de bloques con agua refrigerada.

2001.- Ejecución de inyección de resinas en el hormigón, de inyección de cemento en la cimentación y de reperforación de drenes en el bloque 1 y en el contrafuerte de la margen derecha de la compuerta del aliviadero de la Presa de Portodemouros.

2003.- Inyecciones en el cimiento y en el cuerpo del aliviadero.

El alcance de las actividades que comprendieron esta obra se descompone en:

– Inyección del cimiento del aliviadero mediante lechada de cemento, y posterior reperforación de drenes.
– Impermeabilización mediante campaña de inyecciones de resinas y cosido del aliviadero.

Los trabajos efectuados consisten en la inyección de lechada en cimiento y la reperforación de drenes, correspondientes a la FASE I de los trabajos, y a la FASE II, que abarca la inyección de resinas y el cosido del aliviadero de la Presa de Portodemouros. Con estas actividades se trataba de mejorar las condiciones del aliviadero, evitando en lo posible la entrada de agua en el mismo, con el fin de ralentizar, de acuerdo a la bibliografía consultada y a la experiencia propia, el proceso expansivo existente. Igualmente se aprovecha la actuación para coser el aliviadero, especialmente sus fisuras fundamentales, aumentado la resistencia al deslizamiento de los bloques del aliviadero.

Como resultado de las actuaciones se pretendía contemplar la posibilidad futura de eliminar la cota de seguridad que se mantiene en la presa por temor a que el agua pudiera filtrarse por las múltiples fisuras del aliviadero y llegara a levantar parte del mismo. Además, con la actuación llevada a cabo en el mismo, se recupera su monolitismo estructural.

2010.- Instalación de 16 piezómetros de cuerda vibrante y perforación de nuevos drenes en las zonas en las que las presiones eran mayores.

3.1 *Resultados obtenidos y lecciones aprendidas*

Todas las actuaciones anteriormente descritas encaminadas a disminuir la velocidad de expansión del hormigón o a su mitigación, véase impermeabilizaciones y riego con agua enfriada no han conseguido su objetivo final, es decir, las tasas de expansión han permanecido inalteradas en el tiempo.

Figura 1. Vigas riostras.

Sin embargo, todas aquellas actuaciones que tuvieron como fin último la liberación de tensiones, conseguir el monolitismo de la estructura, impermeabilización de la cimentación, arriotramiento de bloques, etc. han resultado verdaderamente exitosas.

Resultados:

- En cuanto a los trabajos de impermeabilización realizados, cabe destacar que la gran superficie que sigue en contacto con el agua en la cimentación respecto a la altura de la presa, se cree pueda ser el factor que ha impedido una ralentización del fenómeno.
- Los trabajos encaminados al enfriamiento del hormigón, sencillamente resultaron infructuosos, posiblemente por la gran dificultad que supone enfriar mediante riego superficial un volumen tan grande de hormigón.
- El trabajo que mejores resultados ha dado ha sido la realización de cortes transversales en el hormigón. Estos cortes lograron reducir las tensiones en la dirección longitudinal del aliviadero, consiguiendo así una disminución de la velocidad de apertura de fisuras en los planos perpendiculares a los cortes.
- Las inyecciones de fisuras mediante resinas han demostrado una gran efectividad, si bien, al no detenerse el fenómeno que las provoca, son trabajos a realizar de forma periódica.
- Los tratamientos de la cimentación mediante inyecciones de cementos, reperforación de drenes y colocación de piezómetros de cuerda vibrante han funcionado correctamente, aunque como se comenta en el punto anterior, se considera necesario realizar este tipo de trabajos cada cierto tiempo, ya que el la efectividad de estos tratamientos es limitada en el tiempo.

Cabe resaltar por lo tanto que todos aquellos ensayos a escala natural realizados en el aliviadero de Portodemouros encaminados a disminuir o mitigar el fenómeno expansivo del hormigón, tales como impermeabilizaciones o riegos de enfriamiento han resultado tener un rendimiento mucho menor del esperado. Parece por lo tanto lógico pensar que este tipo de tratamientos no tendrán gran efectividad en presas de gravedad, en las que la relación de superficie no impermeabilizable respecto a la que sí lo es es muy alta, y donde los volúmenes de hormigón son realmente importantes.

4 LIBERACIÓN DE TENSIONES MEDIANTE CORTES TRANSVERSALES

Durante el año 1994, y como consecuencia del incremento de tensiones en dirección longitudinal

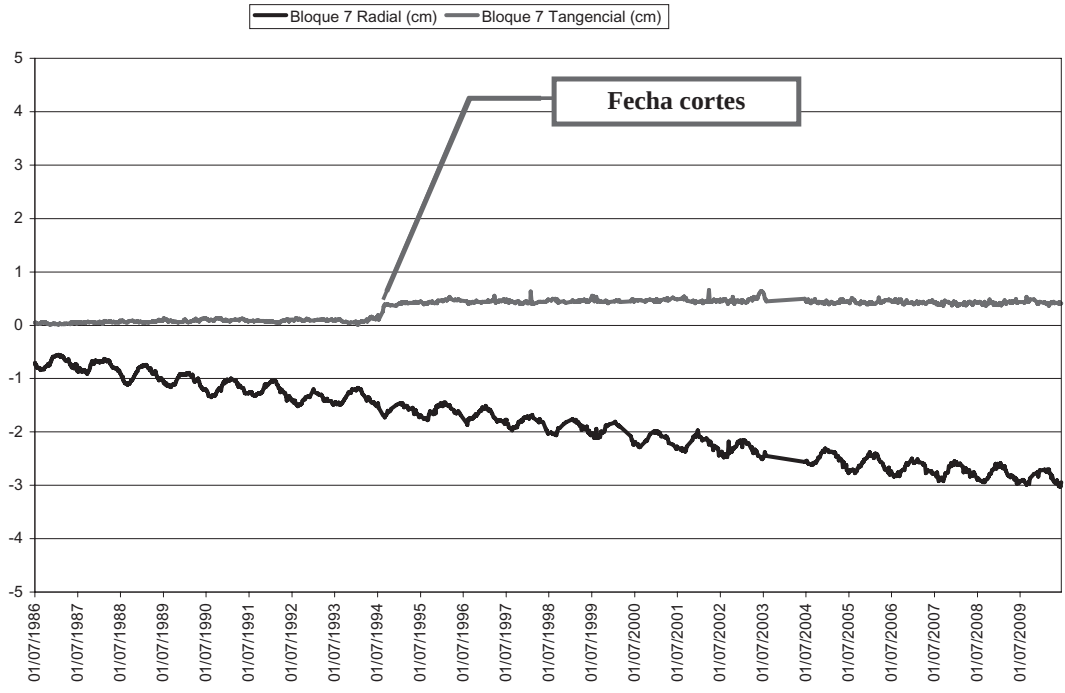

Figura 2. Movimientos bloque 7.

del aliviadero, se acomete la realización de 4 cortes transversales (que luego veremos fueron 5) con el fin de disminuir dichas tensiones. Desde hacía ya varios años, las juntas transversales mostraban un movimiento nulo en apertura y cierre, indicativo claro de su cierre.

Los cortes a realizar (nuevas juntas) se diseñaron en los bloques 3, 5, 8y 10, siendo su grueso 12 mm, excepto en el bloque 5, que fue de 24 mm (el primer corte realizado y que cerró inmediatamente después de terminado). Con el fin de mantener la estanqueidad del conjunto, se rellenaron los cortes con un material muy deformable y se impermeabilizó en galería y paramentos con una junta tórica.

Los 4 cortes se instrumentaron en la galería inmediatamente después de ser realizados, con el fin de poder conocer su comportamiento a lo largo del tiempo mediante juntas de elongámetro.

Como se ve en los gráficos adjuntos, las juntas previamente instrumentadas sufren una repentina apertura al realizar los cortes transversales, moviéndose en apertura y cierre desde la fecha hasta hoy con cierta normalidad, aunque la amplitud va disminuyendo poco a poco.

Los péndulos (fig. 1) indican del mismo modo un claro movimiento de los bloques que los albergan en sentido longitudinal, siendo el del bloque 7 hacia margen derecha y el del bloque 12 hacia margen izquierda.

Las bases de elongámetro de los cortes indican que estos tienen ciclos a apertura y cierre normales en ciclos de invierno-verano, aunque su amplitud es cada vez menor.

5 LA DIFÍCIL TAREA DE RECUPERAR EL NIVEL DE EMBALSE ORIGINAL

No cabe duda de que las acciones acometidas durante todos estos años han ido encaminadas en primer lugar a conseguir unos niveles de seguridad aceptables, y en segundo lugar a restablecer los niveles de explotación originales del embalse.

La experiencia en este tipo de fenómenos nos ha dictado que la realización de los distintos trabajos a acometer de forma dilatada en varios años, no ayuda en absoluto a tomar decisiones. Como la expansión progresa, la fisuración y el deterioro de la cimentación también lo hacen, de forma que no es posible hacer coincidir los mejores niveles de rehabilitación del hormigón y de la cimentación si no se acometen los trabajos a la vez.

Merece la pena realizar un proyecto conjunto de rehabilitación y "primer llenado" inmediato, para así no dilatar la decisión en el tiempo y controlar

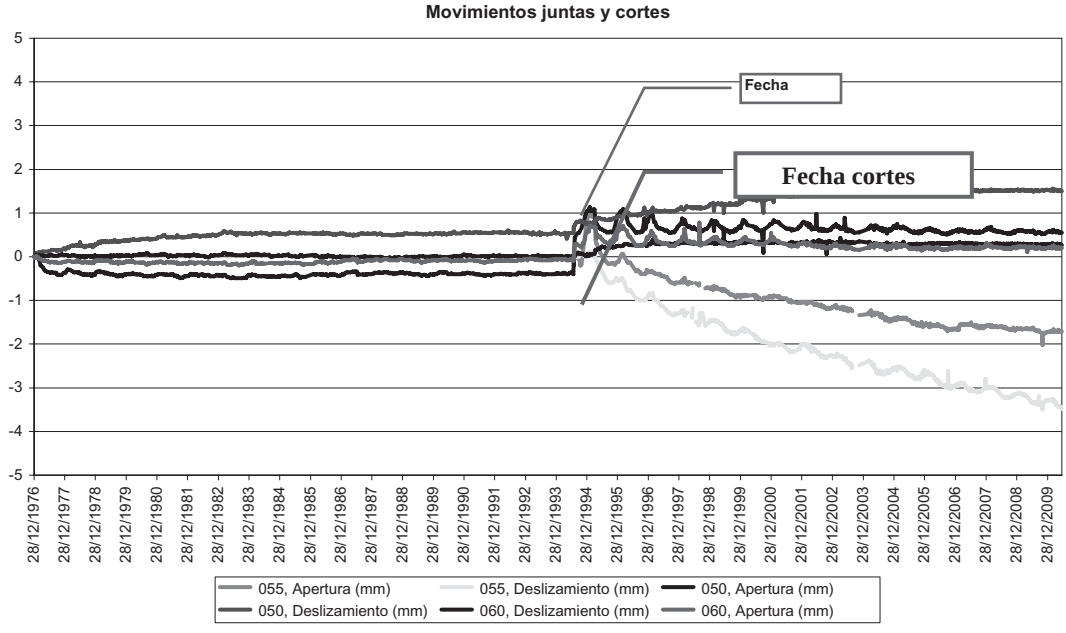

Figura 3. Aperturas-cierres juntas y cortes.

la evolución de la estructura ya puesta en carga, ya que sino, será cada día más y más difícil lograr el objetivo final.

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Rehabilitación integral de la presa de la ribeira. Nuevos métodos para la mejora de la seguridad al deslizamiento, control de subpresiones y sellado de filtraciones

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ABSTRACT: The information we are about to give you, is about a new work methodology, very efficient, which has started to be used with great success in the Ribeira dam, for an optimal testing of the drainage system. On the job, we have questioned some of the concepts relative to sub pressure: waterproof screens, drainage and piezometer net, manage with few scientific base and in many cases have little success. The proposed method starts firstly with the execution of the piezometer net followed by the analysis and readings of the pressure levels, giving a dimension to the three configurations in the drainage network: space, deepness and diameter, taking into account the real conditions of every dam).The use of specially adapted geophysical methods enables the execution of drainage only where they are necessary, eventually including the injections of polymers time to time to reinforce the water resistance of the screens in particularly sensitive zones and where the dam is in use. We can also explain other rehabilitation jobs executed, like the injection of polymers putty slightly expandable at high pressure, used for the sealing of important infiltrations, in diverse fractures of the rock foundation.

1 HISTORIA DE LA CONSIDERACIÓN DE LA SUBPRESIÓN

La rotura de la presa francesa de Bouzey, ocurrida en 1895, fue el detonante para que se introdujera el concepto de subpresión en el cálculo de estabilidad de las presas, aunque se tardaría, sorprendentemente, más de 40 años en empezar a comprender el fenómeno. En un principio se pensó que el empuje del agua bajo la presa se debía a la formación de una grieta en el contacto, por la que entraba el agua, directamente desde el embalse. La teoría de la Grieta, ha llegado hasta los años 80 con bastante pujanza. Sin desdeñar la posibilidad, constatada en numerosos casos, de que aparezcan grietas en el pié de presa de aguas arriba, la subpresión se produce, de forma generalizada, por la naturaleza porosa tanto del hormigón de la presa, como de la roca sobre la que apoya la estructura.

A día de hoy se siguen aplicando antiguas reglas empíricas y obsoletas, y en algún caso manifiestamente erróneas para el cálculo y dimensionamiento del drenaje.

2 INFORME DEL ICOLD EUROPEAN CLUB

Uno de los documentos más clarificadores sobre esta materia es el informe realizado por el ICOLD EUROPEAN CLUB, titulado "Working Group on Uplift Pressures Under Concrete Dams", fechado en 2004 y coordinado por G. Ruggeri. Este informe es de acceso libre y está disponible en la Web de ICOLD.

Son de destacar las siguientes conclusiones del informe.

2.1 *Estado del Arte*

De los estudios recientes analizados por este grupo de trabajo, se desprende que la motivación común de aquéllos, ha sido comprobar que los criterios de diseño universalmente admitidos sobre los efectos del drenaje, de la pantalla de impermeabilización o de cualquier otro método de control de las subpresiones, nunca han sido plenamente contrastados. Muchas presas antiguas, a

la luz de las actuales exigencias de seguridad y de los criterios tradicionalmente admitidos sobre la subpresión, obligan a actuar sobre ellas. Un mejor conocimiento de los mecanismos de la subpresión y de su control, podrían contribuir a obviar innecesarias actuaciones.

2.2 *Pantalla de impermeabilización*

Su profundidad puede variar entre el 10% y varias veces la altura de la presa; y aunque está universalmente admitido, que la correcta ejecución de la pantalla de impermeabilización reduce las filtraciones bajo la presa, su influencia en la disminución de la subpresión es todavía un tema de debate. Efectivamente, aunque pantallas, de dos o tres líneas de taladros de inyección, muy bien ejecutadas, parece que pueden contribuir a disminuir la subpresión, no se les puede confiar, en exclusiva, este cometido.

2.3 *Drenaje*

Todos los estudios confirman que un correcto drenaje es el medio más efectivo para conseguir la reducción de la subpresión.

2.4 *Cambios estacionales*

Los datos confirman que los cambios estacionales de temperatura pueden producir alteraciones significativas en la subpresión; los mayores valores se alcanzan en tiempo frío. Los cambios de temperatura pueden asimismo tener influencia sobre la falta de linealidad de los valores de la subpresión con respecto a la cota de embalse.

2.5 *Medida de la subpresión*

Una conclusión muy importante, sobre la que volveremos más adelante es que la forma, bastante habitual, de medir la subpresión cerrando un dren y colocándole un manómetro es una práctica absolutamente descartable.

2.6 *Subpresiones en el cuerpo de presa*

A pesar de la importancia que tienen para la seguridad de la presa, prácticamente nunca se mide la presión intersticial en el cuerpo de presa.

3 LA PANTALLA DE INYECCIÓN

Aunque forma parte del ideario común que la pantalla de impermeabilización es el elemento esencial de prevención de la subpresión, este concepto es falso. La pantalla de impermeabilización no es estanca, y, en el mejor de los casos, si está correctamente

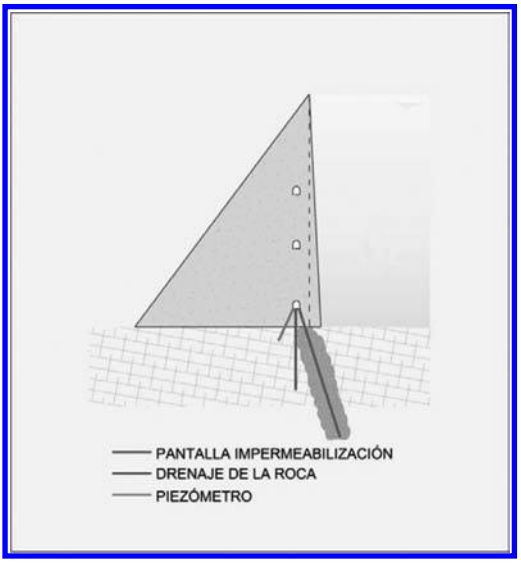

Figura 1. Disposición de la pantalla, drenes y piezómetros.

dimensionada y ejecutada, se limitará a reducir significativamente el caudal de filtración y a aumentar las pérdidas de carga de ésta. Pero que la subpresión se manifieste, independientemente de la pantalla, es una mera cuestión de tiempo.

Por lo tanto, para reducir la subpresión a magnitudes asumibles, es imprescindible eliminar el agua intersticial, tanto del cuerpo de presa, como del cimiento, mediante drenes. Un dren, al igual que lo haría un pozo produciendo un abatimiento del nivel freático, tiene que distorsionar lo máximo posible las líneas de corriente en su entorno. Es evidente que la pantalla de impermeabilización, al reducir las filtraciones bajo la presa colabora, de forma importante a la efectividad del drenaje. En los años 50, algunos autores hablaban del "concierto entre la pantalla de impermeabilización y la red de drenaje".

Vemos, por lo tanto que para el control de las subpresiones se requiere un primer elemento que es la pantalla de inyección, un segundo que es la red de drenaje y un tercero que sería la red de piezometría, donde miden aquéllas. Su disposición ha de ser la indicada en la Figura 1, hacia aguas arriba, la pantalla de impermeabilización, a continuación la red de drenaje, y, hacia aguas abajo la red de piezometría.

4 PROFUNDIDAD DE LOS DRENES

El dren ha de profundizar lo suficiente en el cimiento, como para conseguir que en su zona de

influencia, la subpresión se reduzca aproximadamente al tercio de la ley triangular teórica en su perfil (Figura 2).

Suele ser práctica muy habitual, fijar como profundidad del dren, a priori, un tercio de la cota de embalse en el bloque correspondiente, lo que no tiene ningún soporte técnico. Evidentemente un dren que se quede corto, que no tenga la profundidad adecuada, no captará las filtraciones que superen la barrera de la pantalla de impermeabilización. (Figura 3).

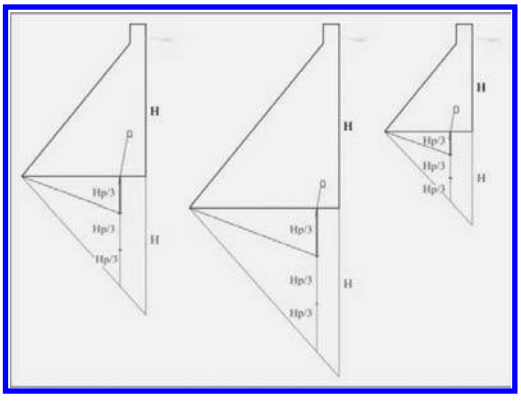

Figura 2. Obtención gráfica de la ley de subpresiones máximas admisibles en cada piezómetro, en tres perfiles.

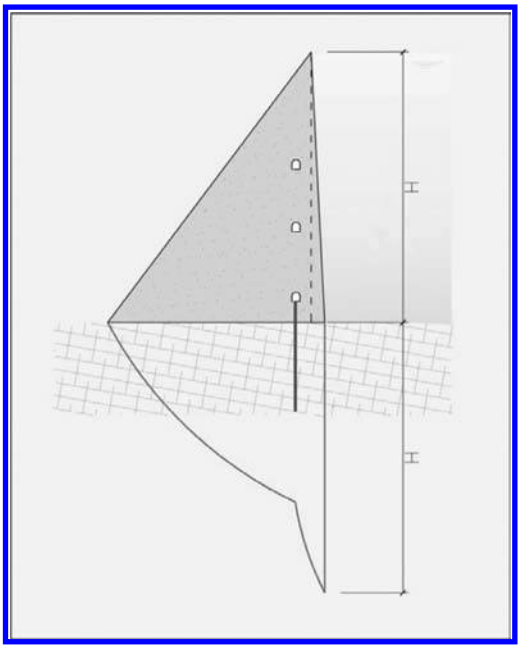

Figura 3. Red de drenaje corta, incapaz de conseguir una disminución adecuada de la ley de subpresiones.

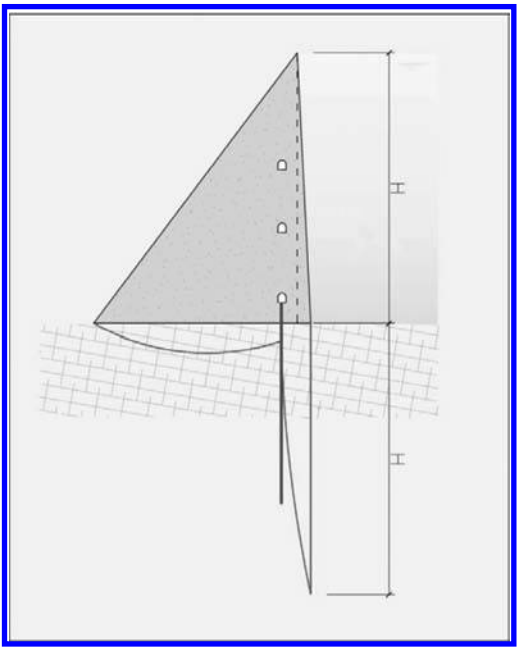

Figura 4. Una red de drenaje eficazmente diseñada es capaz de conseguir una adecuada reducción de la ley de subpresiones.

Evidentemente la situación ideal es la que representamos en la Figura 4, donde los drenes se han profundizado lo suficiente como para lograr una efectiva recogida de las filtraciones, y por lo tanto una correcta disminución de la ley de subpresiones.

El problema estriba en cómo establecer, mediante el cálculo, la profundidad, la separación, el diámetro e incluso la inclinación de los drenes. Desgraciadamente no hemos encontrado aún respuesta, dada las dificultades para poder modelizar la roca y sus diaclasas.

5 NUEVA METODOLOGÍA DE DIMENSIONAMIENTO DE LA RED DE DRENAJE

La nueva metodología que propugnamos parte de una idea muy simple: si la red de drenaje tiene como principal función, rebajar las subpresiones hasta los límites que los cálculos de seguridad establezcan, la primera actuación ha de ser la instalación de la red de piezometría, y su lectura. Evidentemente, antes del primer llenado de una presa es necesario disponer de un mínimo de drenes, pero siempre auxiliados por una completísima colección de piezómetros, a ser posible de cuerda vibrante.

Así, en la presa de La Ribeira, en primer lugar se estableció la posición en alzado de la ubicación de los piezómetros. Se instalaron veintiocho (28) sensores de cuerda vibrante, entre 2 y 3 por bloque. Se ubicaron en el contacto presa-cimiento inclinados hacia aguas abajo de la red de drenaje, con el fin de independizarlos de ésta.

Posteriormente, se dibujaron (Figura 5) todas las secciones de la presa que disponen de un piezómetro. De la Figura 2 se obtuvo la ley triangular teórica máxima en el contacto, justo en la posición de cada piezómetro, de donde se calculó la subpresión máxima admisible en cada sección. A partir de esos valores, se representó en la Figura 5 la curva de subpresiones máximas admisibles y la ley real medida en los piezómetros instalados en la presa de La Ribeira.

Evidentemente, a partir de este esquema, resultó extraordinariamente sencillo determinar las zonas que requerían una mejora del drenaje y las que no necesitaban intervención alguna.

Una vez conocidas las zonas sobre las que había que actuar y la magnitud de las subpresiones a reducir, quedaba la incógnita de cómo lograrlo con la mayor eficiencia. Las variables comprendían el diámetro del dren, su profundidad y la separación entre ellos. La magnitud de la obra a realizar en La Ribeira, donde en amplias zonas de la presa se duplicaban las subpresiones admisibles, nos llevó, de común acuerdo con Endesa, a realizar distintos tramos de pruebas, evidentemente, con el objetivo de rebajar las subpresiones con el menor coste posible.

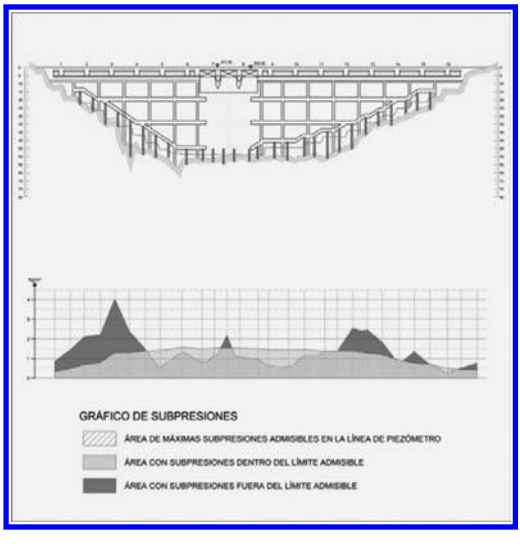

Figura 5. Alzado de la distribución de piezómetros en la presa de La Ribeira y curvas de subpresiones reales y máximas admisibles.

De entrada señalamos que el diámetro del dren, de 76 ó 100 mm., no tiene influencia alguna, por lo tanto, seguimos recomendado realizarlos de 76 mm., que tienen menor coste.

Una vez eliminada una de las variables, quedaban las otras dos, la profundidad y el espaciamiento. Y también una pregunta en el aire, habida cuenta que no se tiene constancia de que durante la construcción de la presa se realizara la pantalla de impermeabilización, ¿se podrían rebajar las subpresiones hasta valores admisibles, sólo ejecutando drenes?

Dado que, a priori no se podía conocer el espaciamiento óptimo entre drenes, se optó por determinar la profundidad óptima, empleando métodos geofísicos.

6 MÉTODOS GEOFISICOS APLICADOS A LA OPTMIMIZACIÓN DE LA RED DE DRENAJE

Los métodos de investigación geofísica, de aplicación habitual en geología y minería, se basan en las diferencias de propiedades como densidad, temperatura, conductividad, permeabilidad magnética, etc., que presentan los materiales que pueden ser analizados en un sondeo o en un pozo.

La idea de utilizar estas técnicas, en las que se combinan geología, física, química y electrónica, para la investigación de drenes, surgió, por un lado al considerar la analogía entre éstos y los pozos, y, por otro, al no lograr, mediante el mero uso de cámaras de TV y el análisis de testigos, toda la información necesaria para la correcta toma de decisiones. El esquema del sistema se representa en la Figura 6.

El agua suele llegar a cada dren por las diaclasas a las que éstos cortan. Generalmente, cuanto más profunda es la diaclasa menor es su aportación de agua debido a la compresión del macizo. La aplicación de métodos geofísicos permite, en primer

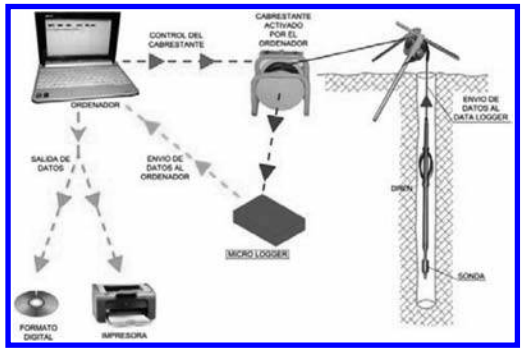

Figura 6. Esquema de los aparatos de geofísica utilizados.

lugar, determinar la posición de estas diaclasas, y en segundo medir su caudal de aportación a cada dren. De esta forma se determina hasta qué profundidad resulta económicamente rentable profundizar los drenes en cada zona.

En la práctica se realiza un dren de gran profundidad en cada zona en la que se subdivida la presa. La profundidad puede llegar al 50% o incluso el 100% de la altura de la presa en ese punto. En primer lugar se introduce una sonda doble, capaz de medir resistividad/conductividad del agua, y su temperatura. El análisis de todo el dren da lugar a un diagrama donde se reflejan, en función de la profundidad, los cambios de estos dos parámetros.

Una vez conocida la posición de las diaclasas, se determinan sus caudales de aportación. Para caudales de cierta importancia, se recurre a una sonda dotada de una hélice vertical, o molinillo, movida por las corrientes ascendentes o descendentes, existentes dentro del dren.

Las medidas se pueden tomar mientras se hace avanzar la sonda a una velocidad constante a través del dren, para generar un perfil continuo del caudal del fluido a lo largo del mismo o, también, se puede fijar la sonda a una determinada profundidad para medir la evolución del caudal en ese punto, y así ver su evolución en función de la cota de embalse o de la influencia de otros drenes cercanos.

De esta manera se determinan con exactitud las profundidades a las que se encuentran las principales vías de circulación de agua lo que permite fijar, de la forma más eficiente, hasta dónde se debe perforar el drenaje en cada zona.

Con el fin de concretar la situación de las zonas de circulación de agua y cuantificarlas, en la presa de La Ribeira se llevaron a cabo seis (6) ensayos geofísicos en sondeos de investigación perforados con las mismas inclinaciones y orientaciones que los futuros drenes para refuerzo de la pantalla de drenaje, alcanzando una profundidad superior incluso al 100% de la carga máxima de embalse.

A modo de ejemplo, en la Figura 7 se representan diagramas de resistividad, temperatura y de

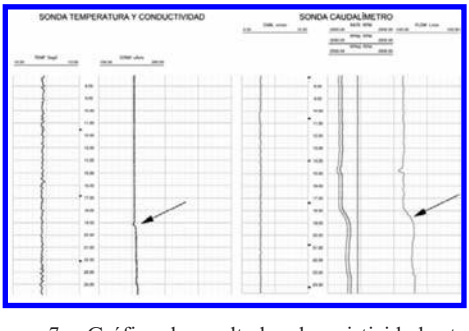

Figura 7. Gráfico de resultados de resistividad y temperatura a lo largo del sondeo en La Ribeira.

aportación de caudales en un sondeo ensayado en la presa de La Ribeira. Un cambio brusco de estos parámetros, delatan la presencia de un aporte de agua.

Del estudio de la gráfica, se pudo constatar, que entre los 18,00 y 19,00 metros de profundidad coincidiendo con la situación del contacto hormigón-roca, existía un importante incremento de caudal de unos 25 l/min que coincidía con un aumento de la conductividad. Además, durante la perforación se detectó ese aumento de caudal, y durante la inspección se pudo comprobar la existencia de una discontinuidad muy próxima al contacto.

7 PROYECTO DE DRENAJE

La red de drenaje, al cabo del tiempo, comienza a perder efectividad, debido tanto a la acumulación de material de arrastre como a la formación de concreciones calcáreas, o que puede llegar incluso a una completa obturación del dren.

HCC ha desarrollado un sistema para la rehabilitación de la red de drenaje que utiliza medios similares a los empleados en técnicas de hidrodemolición, con potentes equipos de bombeo de alta presión que alcanzan las 1.500 atm, combinados con importantes caudales (>100 l/min), que impulsan el agua de limpieza hasta toberas de alta resistencia que son introducidas en el interior del dren.

Los chorros de agua a alta presión, no solamente desincrustan las concreciones calcáreas, sino que penetran en las litoclasas, efectuando una eficaz limpieza tanto de antiguos detritus como de los eventuales arrastres de finos. Además el flujo de agua ascendente arrastra la mayor parte de los sólidos desprendidos hacia la boca del taladro.

Como paso previo a la ejecución de nuevos drenes, se realizó una rehabilitación de zonas concretas de la red de drenaje existente mediante la tecnología, patentada a nivel mundial por HCC, de agua a alta presión y elevado caudal. Rehabilitándose cuarenta y tres (43) drenes hasta la profundidad óptima indicada por los ensayos geofísicos anteriormente ejecutados.

En los bloques que, una vez llevada a cabo esta rehabilitación de la red de drenaje, seguían presentando una subpresión demasiado elevada, se planificó, de acuerdo con Endesa, la ejecución de un determinado número de drenes repartidos a lo largo de la presa. El dimensionamiento de la red de drenaje se realizó teniendo en cuenta nuevamente los resultados arrojados por los ensayos geofísicos.

Una vez conocida la profundidad óptima de perforación en cada zona, sólo quedaba determinar el espaciamiento óptimo entre drenes y su orientación ideal. Para ello, se comenzó con distancias

amplias, de unos cuatro metros. La presencia de los piezómetros y su lectura regular, permitió conocer en tiempo real la eficacia de cada dren en su entorno. La realización de gráficos semanales, permitió proyectar el trabajo para la siguiente semana, disminuyendo la separación entre drenes exclusivamente donde se requirió (Figura 8).

En algunas zonas puntuales, en los bloques de la presa donde la distancia entre la galería y el cimiento ($H_{cont\text{-}gal}$) es elevada, como ocurría en los bloques 2 y 3 de la presa de La Ribeira (Figura 9), la red de drenaje vertical tendrá una eficacia limitada. Los drenes verticales sólo pueden aliviar subpresiones

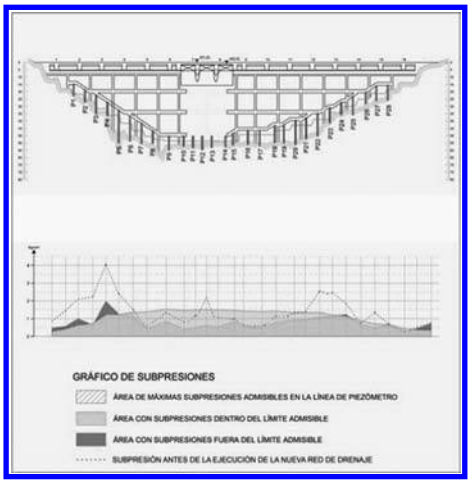

Figura 8. Gráfica de subpresión, una vez completada la red de drenaje. La línea de puntos representa el estado inicial.

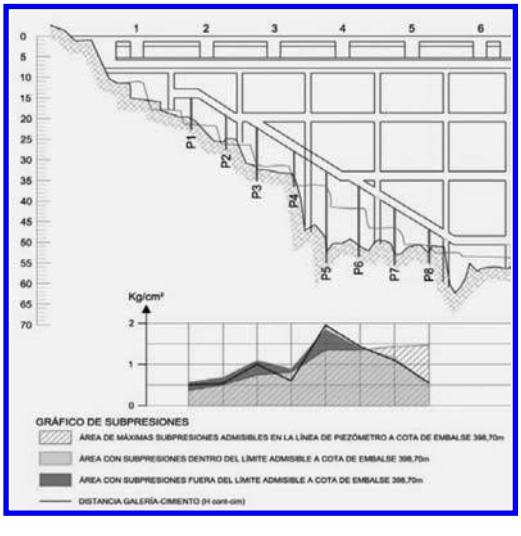

Figura 9. Gráfica de subpresión. La línea de puntos representa la distancia galería-cimiento. ($H_{cont\text{-}cimiento}$).

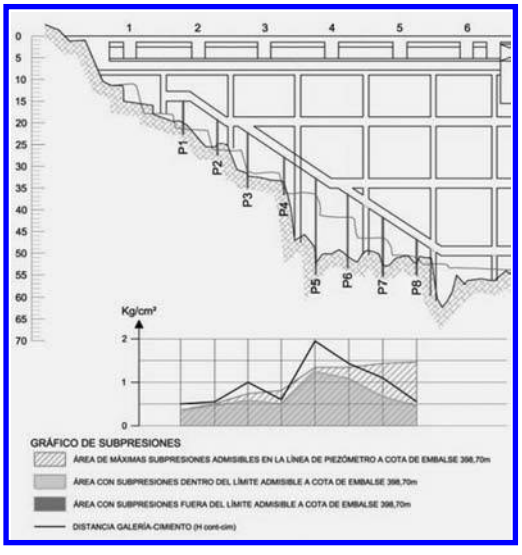

Figura 10. Gráfica de subpresiones registradas en bloques 2, 3 y 4 una vez finalazo el refuerzo de la pantalla de drenaje.

hasta un límite equivalente a aproximadamente la distancia boca-contacto. Con drenes correctamente orientados, las subpresiones descienden hasta límites admisibles, lo que se aplicó en el caso de La Ribeira, con resultados muy satisfactorios (Figura 10).

8 INYECCIÓN LOCALIZADA DE DIACLASAS. NUEVAS RESINAS VISCOEXPANSIVAS

Una vez logrado el objetivo de reducir las subpresiones a valores admisibles, puede suceder que los caudales aportados por los drenes sean altísimos, lo que puede generar arrastres indeseados de material de la cimentación, suponer una pérdida excesiva del agua embalsada o generar la necesidad de disponer de un sistema de achique demasiado potente en el pozo de bombas. De ahí deriva la necesidad de realizar un refuerzo localizado de la pantalla de impermeabilización, con el fin de reducir los caudales de agua circulantes por la roca de cimentación de la estructura.

En el caso de La Ribeira, tras la ejecución de los nuevos drenes de cimiento de los bloques 2, 3 y 4, se constató que, aunque se había logrado una considerable disminución de las subpresiones registradas, en algunos de estos drenes se recogían unos caudales elevadísimos, de hasta 200 l/min, con arrastres de fragmentos de roca y de una sustancia de color anaranjado. Este material se analizó en su momento en laboratorio, resultando ser

de naturaleza inorgánica, con un alto contenido de hierro proveniente probablemente de la propia roca de cimentación, compuesta en esa margen por gneis y pizarras. Esto podría ser indicativo de una degradación paulatina del cimiento como consecuencia de las fuertes corrientes de agua circulantes bajo la presa.

Con el fin de concretar la situación de las zonas de circulación de agua y cuantificarlas, se llevaron a cabo seis (6) ensayos geofísicos en sondeos de investigación perforados con las mismas inclinaciones y orientaciones que los futuros taladros de inyección para refuerzo de la pantalla de impermeabilización, alcanzando una profundidad superior incluso al 100% de la carga máxima del embalse.

Se emplearon sondas de medida de temperatura, resistividad y caudalímetro de alta precisión, con registro en soporte informático en tiempo real de los datos obtenidos. Se detectaron así diferentes zonas de aporte y salida de caudales muy elevados de agua a través de fuertes diaclasados de la roca, que actuaban como vías preferenciales de circulación. (Figura 11).

Tras el análisis de los datos recabados con los ensayos geofísicos, y en vista de los altos caudales aportados por algunos de los drenes de cimiento, contrastados con la información obtenida durante la perforación de los sondeos de investigación y de su inspección con cámara, se elaboró una propuesta concreta de actuación para cada uno de los bloques, que se limitó a las zonas en las que se detectó las circulaciones de agua.

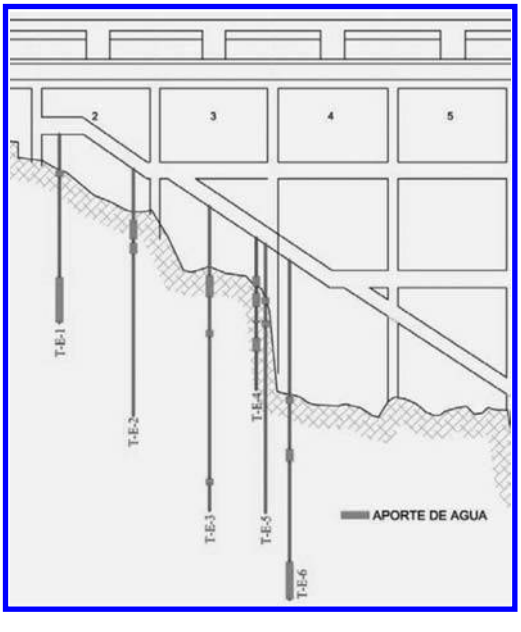

Figura 11. Aporte de agua en los sondeos estudiados en los seis ensayos geofísicos.

Se decidió acometer un refuerzo localizado de la pantalla de impermeabilización del cimiento, buscando reducir de manera significativa los elevados caudales de agua circulantes bajo la estructura, junto con los arrastres provocados por los mismos. De manera secundaria lograr una cierta disminución de las subpresiones asociadas a dichos caudales.

Estos trabajos se ejecutaron con el nivel de embalse lo más alto posible para que los caudales de filtración y subpresiones se manifestarán al 100%, y además, para poder realizar en tiempo real un control de la eficacia alcanzada, pudiendo aforar los caudales aportados por los drenes, registrar los arrastres y medir las subpresiones indicadas por los piezómetros mientras se desarrollaba el tratamiento.

Obviamente, se requería de un material que mantuviera sus propiedades con el tiempo, que no se diluyera en agua, ni fuera arrastrado por ésta, y que no contaminara. Todo ello invalidaba el empleo de materiales de naturaleza cementosa. Por lo que se decidió emplear resinas altamente viscosas, especialmente formuladas para este tipo de trabajos.

Para poder inyectar este material, que no se rige por las leyes convencionales de la hidráulica al no ser newtonianos, se requieren equipos de inyección capaces de dar más de 600 kp/cm^2 de presión en salida de bomba.

HCC en la presa de La Ribeira inyectó de forma controlada resinas epoxis acuaestables, dado que se encontraron numerosas discontinuidades de gran espesor y el volumen a rellenar podía aumentar significativamente el coste económico.

Para dar solución a estos problemas HCC desarrolló un nuevo producto de inyección, inexistente en el mercado, consistente en una resina de alta viscosidad expansiva, que reacciona en presencia de agua o en ausencia de la misma, con capacidad para aumentar hasta 6 veces su volumen original sin transmitir presiones adicionales, pero con una elevadísima viscosidad. Este producto combina las características de las resinas epoxi acuaestables de HCC y las de las resinas convencionales. Se suministra en relaciones volumétricas 1:1, realizándose la mezcla de componentes a la salida de la bomba, bajo alta presión, de manera automática. Además este producto se crea con diferentes grados de viscosidad y capacidad de expansión.

Para el caso de La Ribeira se recomendó el empleo de las siguientes resinas:

– MEDIA VISCOSIDAD: viscosidad media y capacidad de expandir hasta 6 veces su volumen original. (Viscosidad = 50.000 cps)
– ALTA VISCOSIDAD: viscosidad alta y capacidad de expandir hasta 4 veces su volumen original. (Viscosidad > 100.000 cps)

Gracias a estas resinas, en La Ribeira fué posible evitar el arrastre del material por las corrientes de agua y, al mismo tiempo, reducir los consumos hasta unas cantidades razonables, cumpliendo el objetivo de disminuir caudales, inyectándose sólo las zonas problemáticas, no afectando a la explotación del embalse y no produciendo agresión alguna al medio ambiente.

A medida que avanzaba la inyección y se iba creando la barrera, se apreciaba en tiempo real una disminución del aforo de los drenes e, incluso, de la subpresión registrada por algunos de los piezómetros. Con estos resultados se demostró la efectividad de los trabajos ejecutados.

9 CONCLUSIONES

La metodología expuesta, está teniendo una gran acogida entre los responsables de presas, ya que permite, tanto en presas antiguas, como en las que están en su primer llenado, controlar filtraciones y adecuar las subpresiones a los parámetros de seguridad, incidiendo sólo en las zonas que lo necesitan. Con esta metodología, se consigue un diseño a medida de la red de drenaje optima para cada presa y al actuar solo en las zonas necesarias e ir comprobando los resultados en tiempo real, se consigue también un importante ahorro, ya que solo se actúa en las zonas que lo requieren y a la profundidad adecuada.

Los trabajos realizados por HCC en la presa de La Ribeira han aportado las siguientes mejoras significativas:

- Control real de las subpresiones existentes en la zona del contacto presa-roca
- Disminución muy significativa de las subpresiones registradas, llevándolas en la mayor parte de la presa a niveles situados por debajo de los máximos admisibles
- Investigación de la roca de cimentación en las zonas que presentan mayores subpresiones asociadas a grandes caudales de aforo en los drenes, con estudio de su diaclasado, determinación de la situación y cuantificación de las principales vías de agua en la roca, etc.
- Realización de las primeras campañas de inyección de diaclasas en la roca, con sellado de las vías de agua circulantes por las mismas, mediante el empleo de resinas epoxi acuaestables de alta viscosidad y de masillas de polímeros ligeramente expansivos. Análisis completo de subpresión

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Reinyección a alta presión con resinas epoxi viscosas de las juntas verticales de la presa de Susqueda

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ABSTRACT: Auscultation data from Susqueda's vault dam revealed there were relative movements between blocks. To recover the structural monolithism and the ability to transmit the loads to the abutments through the arches, the vertical joints needed to be reinjected. Several tests and the technical-economical analysis of different solutions were carried out. Finally it was decided to use high viscosity epoxy resins, specially formulated with chemical characteristics to support the stress that would be required. A plan of injections by stages was designed to inject joint areas with larger openings first. Boreholes were drilled with rotation cutting equipment from the galleries of the dam. TV inspection equipment were used. The high pressure injections were carried without empting the reservoir while monitoring the movements of the structure. Subsequent the structural behaviour improved and structural monolithism was achieved. To our knowledge this has been the first work of this kind carried out in the world.

1 INTRODUCCIÓN

1.1 *Descripción de la presa*

La presa de Susqueda, cuya construcción finalizó en 1968, se encuentra en la provincia de Gerona, ubicada sobre el río Ter en una cerrada constituida por gneis, dioritas y pórfidos graníticos.

Se trata de una presa bóveda simétrica de doble curvatura, con 135 m de altura máxima sobre cimientos y 510 m de longitud de coronación, la cual está situada a la cota 357,00 m.s.n.m.

Está dividida en 33 bloques de 15 m de anchura, conformando los recintos de inyección entre galerías con los correspondientes elementos de estanqueidad. El espesor de la estructura varía entre los 25 m de la base en su zona más central a los 5 m de la coronación.

Cuenta con siete galerías horizontales visitables, además de la perimetral, a las cuales se accede desde coronación, mediante escaleras y ascensor.

La presa tiene un aliviadero central de labio fijo dividido en siete vanos, con una capacidad de 2.800 m^3/s. Cuenta también con cuatro desagües de fondo de 200 m^3/s de capacidad en total.

Forma un embalse, perteneciente a la C.I. Cataluña, con una superficie de 463 ha y una capacidad a NMN de 233 hm^3, con uso para abastecimiento, riego y aprovechamiento hidroeléctrico.

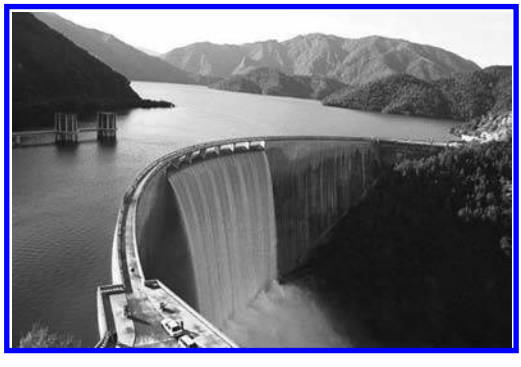

Figura 1. Vista general de la presa de Susqueda.

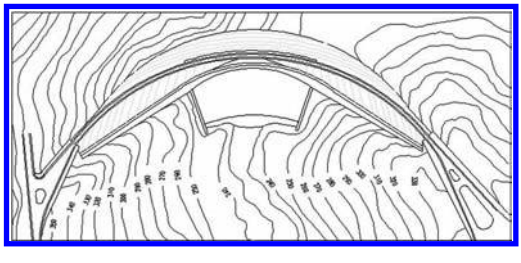

Figura 2. Representación en planta de la presa de Susqueda.

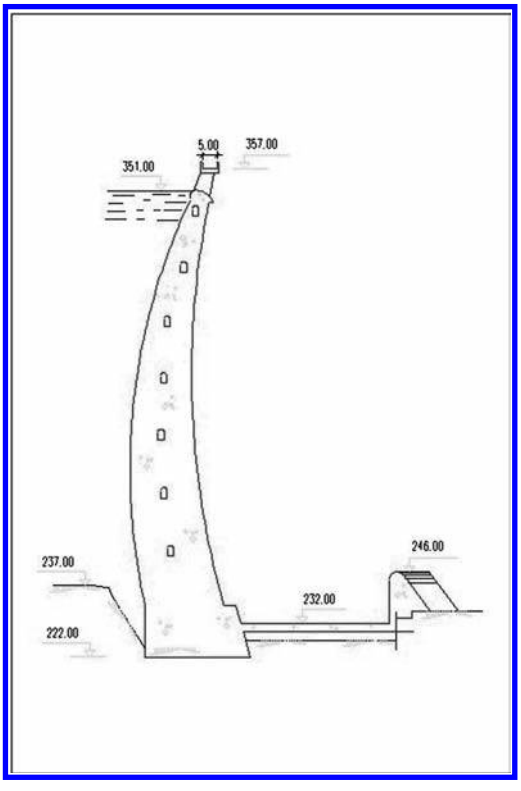

Figura 3. Sección en perfil de la presa de Susqueda.

El titular de la presa es **ENDESA GENERACIÓN**.

1.2 *Antecedentes*

La auscultación de la presa de Susqueda reveló que, sobre todo cuando no existía una carga de embalse alta (inferior a la 320 msnm), la transmisión de esfuerzos compresivos de unos bloques a sus contiguos no tenía la continuidad deseable para una estructura de estas características.

De este modo la apertura de juntas entre bloques presentaba una cierta deriva, lo cuál aconsejaba proceder a la reinyección de las juntas verticales para mejorar la respuesta estructural de la presa a las solicitaciones que recibe, fundamentalmente térmicas y de carga hidrostática, y recuperar su capacidad de transmitir las solicitaciones entre bloques, hasta los estribos.

En los siguientes apartados de esta comunicación se describen las actuaciones realizadas para corregir las patologías que presentaba la presa, la justificación de la solución adoptada y los resultados alcanzados.

2 DESCRIPCIÓN DE LOS TRABAJOS

2.1 *Recopilación y análisis inicial de los datos disponibles*

En el Proyecto de reinyección de juntas se hizo un meticuloso trabajo de recopilación y análisis del historial de la presa, datos e informes disponibles hasta la fecha.

De manera muy resumida, se concluía que:

- El llenado parcial de la presa de 1967, al realizarse con las juntas sin inyectar, provocó que la basculación de la presa hacia aguas abajo se produjera con los arcos superiores soportando más carga de la prevista.
- La inyección de las juntas, acometida en el primer cuatrimestre de 1968, se realizó cuando el hormigón de la presa se hallaba aún en la fase de pérdida del calor de hidratación. Señalar que hasta 1972 no puede darse por concluido este proceso, produciéndose en este periodo unos descensos térmicos notables. Esto provocó unas aperturas de juntas adicionales con posterioridad a su inyección.
- En el comportamiento de las juntas influyen sobre todo solicitaciones debidas a las variaciones térmicas y al nivel de embalse (a niveles bajos se producen mayores aperturas de juntas). La evolución es negativa, produciéndose la apertura de las juntas con niveles de embalse cada vez más altos.

Análisis de comportamiento previos de la presa recomendaban proceder a la reinyección de las juntas de contracción de la estructura. De acuerdo con dichos estudios, se decidió la realización de un proyecto de inyección que definiese con detalle método y épocas más apropiados para la ejecución de la inyección.

Para definir la reinyección, se analizaron los registros históricos de movimientos de juntas, fijándose en función de los mismos la época del año y los niveles de embalse idóneos para acometer los trabajos en cada zona de la presa. Ver Figura 4.

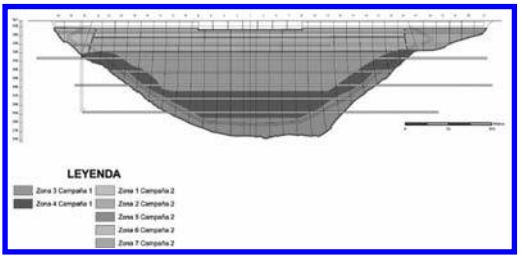

Figura 4. Alzado de la presa de Susqueda con zonas y campañas de reinyección de juntas verticales.

Los trabajos se desarrollaron entre los años 2004 y 2006.

2.2 *Estudio de la apertura y cierre de juntas*

Con el fin de complementar los datos disponibles y poder conocer de manera exhaustiva los movimientos de apertura y cierre de cada uno de los recintos en que se dividen las juntas verticales, al sistema automático de adquisición de datos de Susqueda se añadió en obra otro portable, formado por bases elangométricas y relojes comparadores, instalados en las galerías, aguas arriba y aguas abajo. Ver Figuras 5 y 6.

Figura 5. Detalle de reloj comparador y terna de base elangométrica instalados sobre junta 8–10.

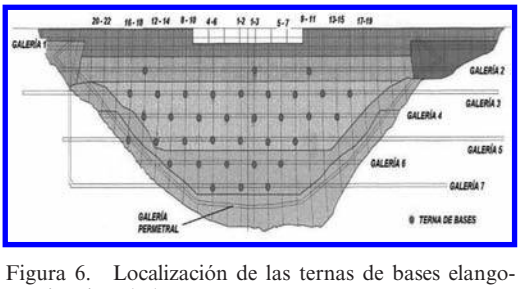

Figura 6. Localización de las ternas de bases elangométricas instaladas.

El seguimiento permanente de las aperturas de todos los recintos, con dos elementos de medida en cada unos de ellos, mediante dos lecturas semanales durante una anualidad completa, permitió optimizar el momento de inyección de cada zona, ajustando las previsiones iniciales. Como efecto secundario positivo, se obtuvo un registro amplio de los movimientos experimentados por la presa con motivo de los trabajos de inyección.

Efectivamente, con un buen conocimiento del comportamiento de la presa se pudo establecer un programa de inyecciones que aprovechó al máximo los propios movimientos de la estructura para tener cada recinto lo más abierto posible durante el trabajo de inyección.

Se elaboraron informes semanales y un informe final anual que reflejaban las mediciones efectuadas a los relojes comparadores y bases elangométricas, incluyéndose gráficas con la evolución de movimientos registrados en cada caso. Ver Figura 7.

En definitiva, y como ya se ha indicado, el estudio previo realizado, así como el seguimiento constante de las deformaciones, temperaturas y cargas de embalse, permitieron elaborar un calendario de intervención extraordinariamente ajustado a los momentos óptimos de inyección.

2.3 *Reinyección de juntas verticales de la presa de Susqueda*

2.3.1 *Perforación de taladros de reinyección*

En un principio se pensó realizar, si no toda, al menos sí parte de la reinyección aprovechando los inyectores originales, idea descartada rápidamente dadas las características y estado de tales conductos, en su mayoría obturados por la lechada de cemento primitiva y provistos de válvulas inservibles (incluidas de origen) y, por tanto, con escasas posibilidades de recuperación. Ver Figura 8.

Se planteó a continuación realizar nuevos conductos de inyección perforándolos en el plano de la junta, método que tras las primeras pruebas también

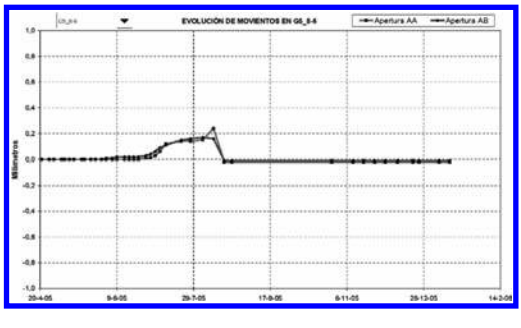

Figura 7. Ejemplo de evolución de movimiento registrado por el par de relojes comparadores (aguas arriba y aguas abajo) en el recinto de una junta vertical.

Figura 8. Detalle del deficiente estado de los antiguos conductos de inyección y purga.

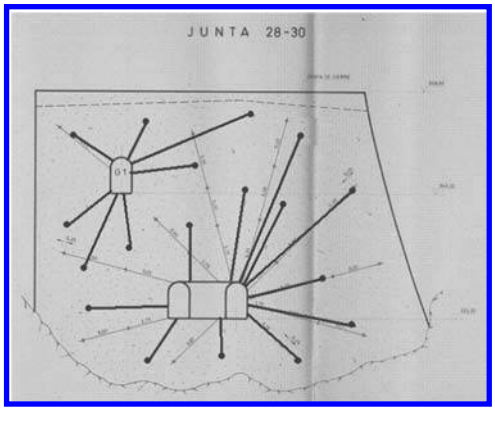

Figura 9. Croquis de la perforación planteada en uno de los recintos.

fue desechado, pues por el alabeo de las juntas el taladro perdía el plano de éstas rápidamente.

Finalmente, se decidió realizar la inyección por taladros secantes a las juntas, ejecutados desde las galerías, con una distribución uniforme en cada recinto para cubrir toda el área a inyectar. Ver ejemplo en Figura 9.

Tal y como se especificaba en el Proyecto, los taladros se perforaron a rotación, con extracción de testigo, empleando corona de diamante y en diámetro comercial de 46 mm, con diferentes longitudes e inclinaciones. El hecho de que los trabajos se realizaran desde el interior de las galerías y de que los taladros tuvieran unas profundidades de hasta 20 m en algunos casos, requería el empleo de una maquinaria de reducidas dimensiones, al mismo tiempo que de potencia suficiente. Ver Figura 10.

El realizar los taladros a rotación, con recuperación de testigo, permitió conocer el estado de la junta intersecada, comprobando de este modo el estado de la misma (posición real, si estaba cerrada o abierta, grado de relleno de lechada de cemento, etc). Esta información se vio complementada por la inspección del interior de una parte de los taladros con microcámaras de TV en color con profundidad sobreimpresionada. Ver Figuras 11 y 12.

Figura 10. Perforación a rotación de taladros de inyección. Puede apreciarse la versatilidad para trabajar en espacios reducidos de la maquinaria empleada.

Figura 11. Testigo obtenido con la perforación de un taladro de reinyección. Se comprueba que la junta se encuentra abierta, con un relleno deficiente de la misma por parte de la lechada de cemento original.

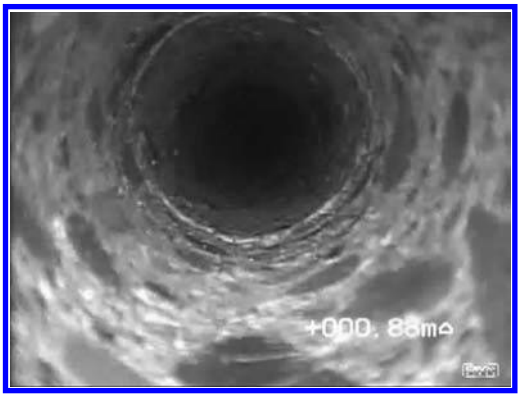

Figura 12. Fotograma de vídeo grabado durante la inspección con microcámara de TV del interior de uno de los taladros de reinyección.

El estado de las juntas observadas era muy variable, con juntas despegadas, otras con relleno de cemento en apariencia insuficiente, e incluso con juntas unidas.

Se ejecutaron más de 1.500 taladros repartidos por toda la presa, con una longitud total de perforación superior a los 6.000 m.

2.3.2 *Elección del material de reinyección*

En el Proyecto de reinyección de juntas se establecía el microcemento como material de inyección a priori, sin descartar el empleo de resinas epoxi capaces de polimerizar en presencia de agua.

Dada la información obtenida en la perforación de los taladros de reinyección, y teniendo en cuenta la variabilidad del estado de las juntas, incluso en un mismo recinto, se decidió el desarrollo de una resina epoxídica viscosa especialmente formulada para el trabajo a realizar, dotada de características físico-químicas acordes con las solicitaciones a las que iba a estar sometida. Ver Figura 20.

A priori se buscaba un material cuyas características fundamentales fueran:

- Precio más bajo posible.
- Resistencia a compresión equiparable, al menos, a la del hormigón.
- Capacidad de polimerizar en seco o en presencia incluso de una corriente de agua.
- No arrastrable por el agua.
- Baja adherencia al hormigón seco y húmedo.
- Módulo elástico a compresión y tracción bajo, que permitiera una cierta deformación del producto con capacidad de recuperación.

Se inició el proceso con seis formulaciones distintas, después de rechazar otras cuatro por incompatibilidades entre los componentes de la formulación.

Tras varias pruebas se fue acotando el campo de ensayo y, finalmente, se seleccionaron dos, que fueron sometidas a ensayos de tracción, flexotracción, compresión y adherencia para verificar que respondían a los requisitos establecidos. Ver Figuras 13, 14 y 15.

Figura 13. Rotura de probeta a flexotracción.

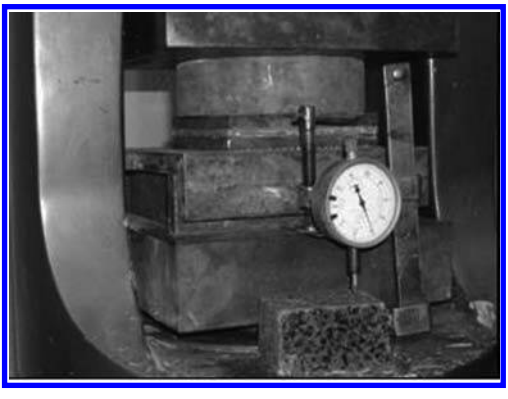

Figura 14. Ensayo de sándwich a compresión.

Figura 15. Ensayo de adherencia resina-hormigón.

Estos ensayos sirvieron para tener una orientación de resultados y seleccionar el material. Posteriormente, y una vez decidido cuál sería la formulación para la inyección, se llevaron unas probetas al Laboratorio de Materiales de Construcción de la E.T.S.I. de Caminos de Madrid para la caracterización del material. En la tabla 1 se resumen las principales características mecánicas obtenidas a partir de los ensayos allí realizados. Se obtuvieron también las curvas de deformación. Ver Figura 16.

En resumen, podía concluirse que:

- La resistencia a compresión, de 422 Kp/cm^2, es del orden de la de un muy buen hormigón, y notablemente superior a la de la lechada de cemento y/o microcemento.
- El módulo elástico es del orden de 100 veces inferior al del hormigón.
- Superada la carga de rotura, el material, aunque soporta un cierto grado de fisuración, plastifica.

Una vez definido el material, y antes de decidir el empleo de microcemento o resina, se decidió realizar sendas pruebas de inyección con este último material en dos recintos de la presa.

Tabla 1. Principales características mecánicas de la resina **HCC-IJ-BV** desarrollada específicamente para la reinyección de las juntas verticales de la presa de Susqueda.

Variable	Valor medio
Resistencia a tracción directa	7,5 N/mm^2
Resistencia a flexotracción	9,9 N/mm^2
Resistencia a compresión	42,2 N/mm^2
Módulo de elasticidad a compresión	323,6 N/mm^2

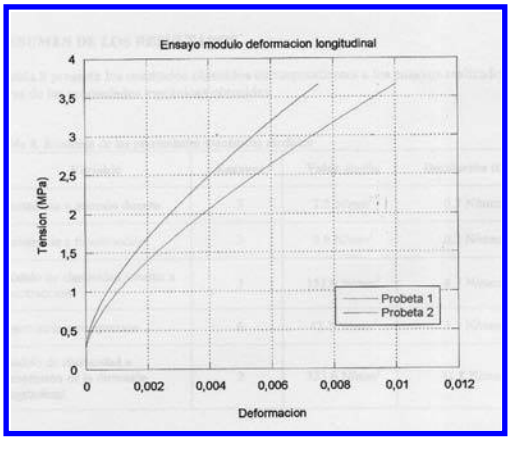

Figura 16. Curvas de deformación obtenidas con los ensayos realizados.

2.3.3 *Prueba de reinyección en dos recintos*

Al mismo tiempo, se pretendía poner a punto la metodología de trabajo a seguir para el caso de que finalmente se decidiera emplear este material en todas las juntas verticales de la presa. Para ello, se elaboró una Instrucción de Trabajo, indicando la metodología a seguir, parámetros de inyección, controles durante la misma, etc.

Con el fin de ahorrar material de inyección, se decidió obturar los taladros en las inmediaciones de la junta a reinyectar.

Previamente a la inyección se realizaron pruebas de agua con el fin de determinar posibles admisiones, comunicaciones entre taladros, etc, y realizar además un lavado de las juntas para eliminar suciedades, restos de lechada degradada, etc, con el objeto de favorecer una óptima penetrabilidad del material y la recuperación de la función transmisora de solicitaciones de la junta.

Dada la relativamente elevada viscosidad del material empleado, del orden de 20.000 cps, las presiones de inyección en boca de taladro son sensiblemente superiores a las necesarias en caso de inyectar lechada cementosa. Sin embargo este hecho no tiene relevancia para la seguridad de la estructura, por dos motivos principalmente:

- Al tratarse de un fluido no newtoniano este polímero presenta unas pérdidas de carga elevadísimas.
- La inyección se produce de manera muy localizada, con lo que la afección a la estructura por la presión de trabajo empleada es enormemente limitada.

Lo que se hizo fue controlar los movimientos de las juntas durante su inyección, estableciendo limitaciones a la apertura de la junta tanto en el recinto propiamente inyectado como en los adyacentes, además de en las juntas contiguas (ver tabla 2). Para ello se instalaron relojes comparadores de precisión de centésima de milímetro. También se instalaron manómetros en los taladros contiguos al inyectado y en los de los recintos adyacentes.

Además, con el fin de disponer en todo momento de información en tiempo real del control de la inyección sobre cada recinto, se instaló un sistema

Tabla 2. Limitaciones establecidas para la apertura de juntas en los recintos reinyectados y en los adyacentes.

Recinto	Apertura máxima (décimas de mm)
Reinyectado	5
Adyacente a izquierda y/o derecha	2
Adyacente arriba y/o abajo	2
Adyacente a los adyacentes a izquierda y/o derecha	1

de comunicación entre los diferentes tajos, situados en distintas galerías, por medio de genéfonos. Ver Figura 17.

Tras la reinyección de los dos recintos, se ejecutaron taladros de control para comprobar la eficacia y resultado del tratamiento aplicado. Se obtuvieron testigos muy interesantes, con las juntas completamente rellenadas de resina y unidas, lo que indicaba que habían recuperado su función estructural. Ver Figura 18.

A partir de la información recogida durante la inyección (apertura de junta, comunicación entre taladros, pruebas de agua y consumos) más los datos obtenidos de los taladros de control, se realizaron esquemas de distribución estimados de la resina en cada uno de los dos recintos inyectados. Ver Figura 19.

Los resultados de esta prueba fueron muy satisfactorios, y llevaron a la decisión de reinyectar todos los recintos de las juntas verticales de la presa empleando la resina específicamente desarrollada para estos trabajos.

En efecto, aunque la utilización de lechadas cementosas en este tipo de actuaciones es perfectamente válida (hay sobrados ejemplos), quedó de manifiesto que en el caso particular de la presa de Susqueda el empleo de esta nueva

Figura 17. Detalle de genéfono empleado durante los trabajos de reinyección para comunicación entre tajos.

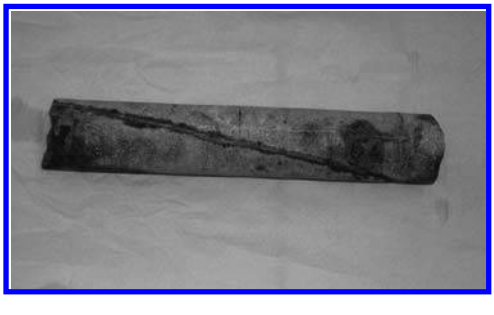

Figura 18. Testigo de junta reinyectada durante la prueba con resina epoxi.

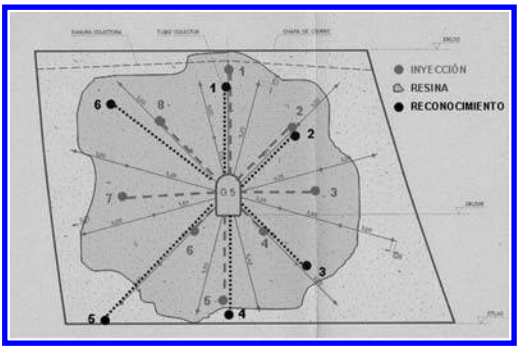

Figura 19. Croquis del resultado de la reinyección de uno de los recintos de prueba.

resina aportaba notables ventajas de índole técnico y, además, de practicidad en la puesta en obra del material, lo que finalmente redundaba en un menor coste de las operaciones y compensaba por completo la diferencia de precio inicial entre este producto y el microcemento. A continuación se detallan las principales de dichas ventajas:

- Superiores características mecánicas, detalladas en la tabla 1.
- El sistema tradicional con lechada cementosa obligaba en el caso de Susqueda al llenado con agua de numerosos recintos cercanos al reinyectado, a controles exhaustivos de movimientos, a equipos de sellado de bandas de estanquidad en alerta permanente, etc, algo que no sucede con esta nueva tecnología de reinyección con resinas epoxi. Esto, además de suponer una ventaja desde el punto de vista logístico y económico de la obra, hacía más sencillo variar, en caso necesario, el programa de obra para adaptarse a circunstancias cambiantes (incremento no previsto de la cota de embalse, por ejemplo).
- Con una reinyección tradicional, el proceso afectaría a toda la estructura debido a las importantísimas fuerzas movilizadas. Sin embargo, con este nuevo método, el proceso es local, afectando exclusivamente al recinto que se inyecta en cada momento, con una repercusión prácticamente nula sobre los contiguos, como se aprecia en el seguimiento del control de aperturas.
- En una inyección a alta presión de resinas se consigue que el material penetre por donde no entra la lechada; exige mayores presiones de inyección pero menores fuerzas aplicadas; su distribución dentro del recinto se controla mejor; hay menor posibilidad de fugas hacia lugares no deseados (retomas, coqueras lejanas, juntas de estanqueidad rota, etc); el cierre mecánico y la impermeabilización

son superiores, y su duración en el tiempo, estimada por las características del material, es mayor.

2.3.4 *Reinyección del resto de recintos*

De acuerdo con lo expuesto, y dado el resultado de la prueba de inyección realizada, se decidió acometer la reinyección de las juntas verticales de la presa empleando el material y metodología ya descritos en los apartados anteriores. Ver imagen de un momento de la reinyección en la Figura 20.

Se comprobó a lo largo de toda la obra que, de esta forma, sólo se rellenaban las zonas que lo requerían, sin introducir grandes fuerzas en la estructura al inyectarse el material de manera muy localizada. En cada recinto se limitó la apertura máxima admisible durante la reinyección con los valores establecidos durante las pruebas practicadas. Como quedó de manifiesto durante la ejecución de las mismas, a diferencia de lo que ocurre en las inyecciones con lechadas cementosas, al abrirse la junta algunas décimas en la zona de inyección no se detectaban movimientos apreciables ni en las juntas adyacentes ni en los recintos superior o inferior del inyectado, donde también se habían establecido las limitaciones de apertura ya conocidas.

Durante la reinyección se llevó a cabo un completo registro de los movimientos. Se extendió a toda la presa el sistema empleado en las pruebas de comunicación entre galerías mediante genéfonos, dado que fue muy efectivo y útil para contar en todo momento y de manera instantánea con la información necesaria en el punto de trabajo.

Se ejecutaron en diferentes recintos taladros de comprobación con el fin de ir controlando la eficacia de los trabajos realizados. Se obtuvieron muchos testigos de las juntas correctamente reinyectadas con la resina empleada, lo que venía a confirmar la bondad del tratamiento aplicado. Ver Figura 21.

Se reinyectaron todos los recintos de la presa, empleándose más de 45.000 kg de resina en el conjunto de la actuación.

Durante toda la obra se llevó a cabo un exhaustivo registro documental de todas las fases de la misma (histórico de movimientos de apertura y cierre de las juntas con el sistema portable instalado, datos de los taladros perforados, pruebas de agua practicadas, resultados de la reinyección de todos los recintos, registros de movimientos, etc). Ver ejemplo en Figura 22.

Una vez finalizados los trabajos, se procedió a restituir las condiciones de limpieza existentes en la presa antes del inicio de los mismos. Para ello se realizó un lavado de todas las galerías con agua a presión. Todos los residuos generados fueron clasificados según su naturaleza y tratados medioambientalmente por un gestor autorizado, cumpliendo la normativa vigente. Ver Figuras 23 y 24.

Figura 20. Trabajos de reinyección de un recinto.

Figura 21. Testigos de juntas verticales reinyectadas con la resina epoxi empleada, completamente rellenadas de material.

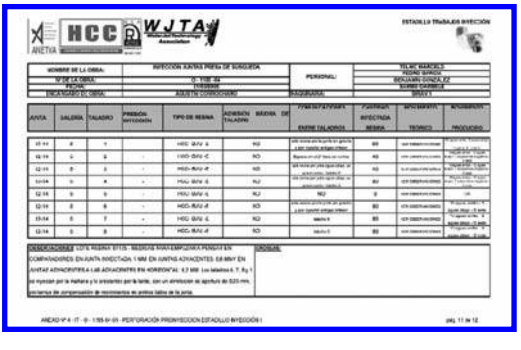

Figura 22. Ejemplo de estadillo de reinyección rellenado en obra.

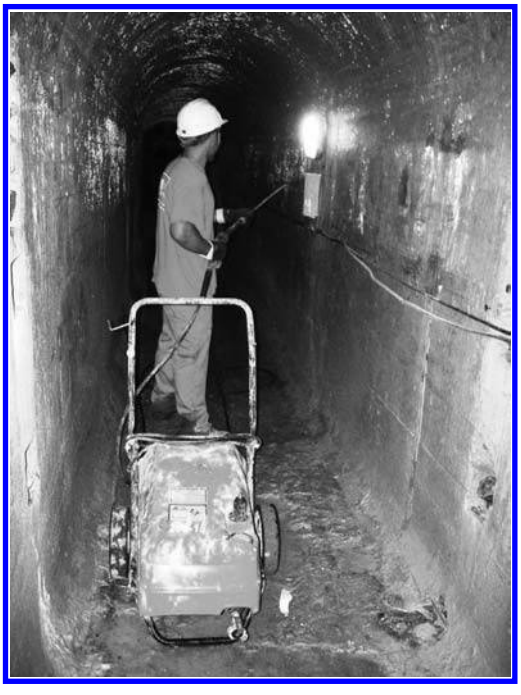

Figura 23. Hidrolimpieza de las galerías de la presa.

Figura 24. Acopio y clasificación de residuos generados en la obra.

2.4 *Resumen*

La medición de las principales unidades de obra (1.500 taladros ejecutados, 6.000 m de perforación y 45.000 kg de resina) da fe de la magnitud de la actuación llevada a cabo, que se prolongó durante varias anualidades, cobrando aún mayor interés por ser, posiblemente, la primera reinyección de juntas verticales en presa bóveda que se lleva a cabo en el mundo empleando esta tecnología.

La reinyección de las juntas se llevó a cabo con la instalación en servicio, pudiendo mantener la cota de embalse a los niveles óptimos para la reinyección de cada campaña, sin tener que bajarla más de lo estrictamente necesario.

3 CONCLUSIONES

Una de las aplicaciones más novedosas de la inyección de polímeros en presas, es, sin duda, la reinyección de juntas verticales en una presa bóveda, como ha sido el caso de Susqueda.

La metodología aplicada y el material empleado, todo ello desarrollado de manera exclusiva para este trabajo, han tenido unos resultados altamente satisfactorios, como demuestra el hecho de que los datos actuales del comportamiento global de la estructura con respecto a los históricos indican que ha mejorado sustancialmente el monolitismo de la presa.

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Rehabilitación de la presa de El Embocador

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RESUMEN: La presa de El Embocador, situada en el río Tajo aguas arriba de Aranjuez, se construyó a principios del siglo XVI bajo el reinado de Carlos I. La estructura original consiste en un entramado de madera, relleno de piedra, impermeabilizado con un tablestacado frontal y cimentado sobre pilotes. En su larga historia ha sido objeto de varias reparaciones y adaptaciones (las últimas en el siglo pasado para su aprovechamiento hidroeléctrico). La impermeabilización del paramento de aguas arriba estaba muy deteriorada por el paso del tiempo y permitía la circulación de agua por el interior del cuerpo de presa, lo que había producido el levantamiento y rotura local del recubrimiento superficial.

El proceso de restauración efectuado ha consistido fundamentalmente en el sellado e impermeabilización del paramento de aguas arriba y en el blindaje con una losa de hormigón armado del de aguas abajo. Tiene como aspecto más innovador la ejecución de una pantalla continua tipo velo mediante técnica de superjet grouting. La plataforma de trabajo necesaria para la construcción de la pantalla se ha ubicado sobre el propio cuerpo del azud, lo que ha permitido trabajar con agua en el embalse, manteniendo la presa en servicio y simplificando notoriamente el sistema de desvío del río.

ABSTRACT: El Embocador dam is located on Tajo River, upstream Aranjuez. It was constructed in the first half of the 16th century, during the reign of Carlos I. The original dam structure consists on a timber framework, filled with rocks, piled, and waterproofed with sheetpiles. Along its long history, it has been object of several updates and adaptations, the last ones at the beginning of the past century for its hydroelectric use. The upstream waterproofing was broken in several points, thus water penetrates into the dam body causing internal erosion and deterioration of the external protective shell.

Rehabilitation works that have been carried out consists on the construction of a frontal waterproofing screen and the lining with reinforced concrete of the downstream face. The most innovative aspect is the execution of the upstream cutoff wall with superjet grouting techniques. This wall has been done using the surface of the dam as working platform and that has made possible: working with water in the reservoir, keeping the dam in service and simplifying the river diversion.

1 INTRODUCCIÓN

La presa de El Embocador está situada en el río Tajo aguas arriba de Aranjuez. La estructura original se construyó entre 1530 y 1534, durante el reinado de Carlos I, para la puesta en riego de la fértil vega existente en la zona. Se trata de un azud de derivación de unos 5 m de altura y 158 m de longitud de coronación, que permite elevar el nivel de las aguas del Tajo para que puedan entrar a los canales de Las Aves (en la margen izquierda) y la Azuda y el Caz Chico (en la derecha).

La tipología y forma de construcción del cuerpo original de la presa (al ser una estructura antigua ha sido objeto de diversas modificaciones durante su explotación) es la habitual de la época en este tipo de azudes. Son estructuras de pequeña altura, en las que el tamaño estaba limitado por las técnicas constructivas y condicionado por su uso, que era básicamente elevar la lámina de agua en el río para derivar caudales, bien para el riego (como es el caso de la presa de El Embocador) o bien para el accionamiento hidráulico de molinos. El cuerpo de presa está constituido por un entramado de traviesas de madera relleno de escollera ripiada con

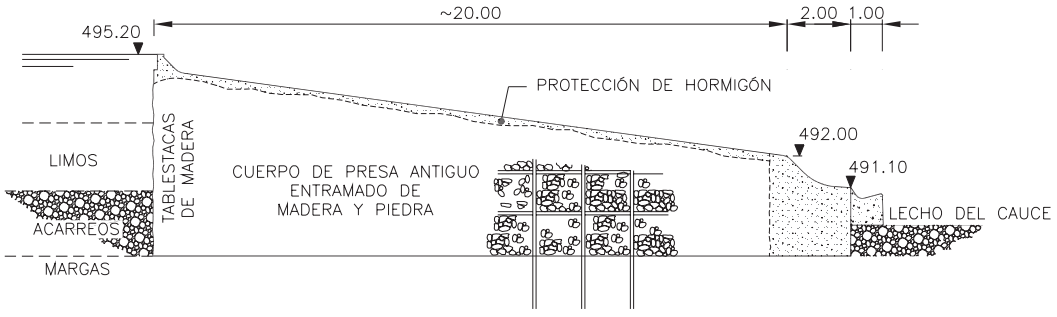


Figura 1. Sección tipo de la presa al comienzo de las obras.

gravas y pilotado en su base. La impermeabilización del paramento de aguas arriba se conseguía mediante una pantalla de tablestacas de madera.

Desde su construcción la presa de El Embocador se ha mantenido en explotación, por lo que ha sido objeto de diversas intervenciones. Las más relevantes han consistido en la colocación de sucesivas capas de protección sobre el cuerpo de presa primigenio (fundamentalmente losas de hormigón o mortero, y también parches realizados con hormigones asfálticos y/o mallas de alambre) y la construcción de sendas centrales hidroeléctricas a comienzos del siglo XX, en los estribos derecho e izquierdo de la presa, para el aprovechamiento del salto generado por el azud.

Las obras de rehabilitación acometidas eran necesarias para proteger la presa del desgaste que se ha producido con el paso del tiempo. Las tablestacas de madera de la pantalla frontal de impermeabilización estaban rotas, o desaparecidas, en muchos puntos por los que se habían creado vías preferentes de entrada de agua al cuerpo de presa, que terminaban (por la acción combinada de la erosión interna y la externa por el tránsito de caudales sobre el azud) levantando y fracturando la losa de protección con que estaba dotado (ejecutada con morteros y hormigones de escasa calidad). El crecimiento de vegetación en el paramento de aguas abajo suponía una dificultad adicional, al complicar los accesos y el mantenimiento de la superficie del azud.

A la vista de la situación en la que se encontraba el cuerpo de presa (en algunos puntos había comenzado a sifonar) y tratándose El Embocador de una estructura que tiene una función importante (actualmente da servicio a los canales de Las Aves, la Azuda y Caz Chico, que sirven una superficie de riego de unas 6.000 ha) y que debe ser preservada por el interés histórico que atesora, la Confederación Hidrográfica del Tajo decidió acometer las obras de reparación y adecuación de la misma, las cuales fueron llevadas a cabo por la empresa FCC Construcción.

Las actuaciones han consistido en la limpieza y retirada de la vegetación del espaldón y vaso de la presa, el picado y saneo de la capas de protección existentes, la realización de una pantalla de impermeabilización en el paramento de aguas arriba, la ejecución de una losa de hormigón armado de 0,30 m de espesor para blindaje del cuerpo de presa, la protección del pie de presa con escollera gruesa y la adecuación de los canales de la antigua central hidroeléctrica de margen derecha para su uso como desagües de fondo. En esta comunicación se describen los trabajos realizados, resaltando los correspondientes a la pantalla de impermeabilización, tipo velo, ejecutada con superjet grouting, que combinada con la solución adoptada para el desvío del río permitió que se pudiesen ejecutar las obras manteniendo la presa en servicio durante la campaña de riegos.

2 CARACTERÍSTICAS GEOLÓGICAS DE LA CERRADA Y DEL VASO

La vega del Tajo, en el entorno de Aranjuez, se caracteriza por ser un valle de gran amplitud, cuyo fondo está ocupado por materiales del Cuaternario (acarreos, gravas y arenas) y las laderas constituidas por materiales del Mioceno con la presencia de yesos, margas y arcillas, intercalados.

El azud de El Embocador se ubica sobre estos terrenos del Cuaternario. Se trata de una zona de morfología suave, cuyo rasgo distintivo es la presencia de terrazas aluviales de gravas y arenas. Son materiales con una capacidad portante limitada, pero suficiente para resistir una estructura de estas características: de pequeña altura, que no está sometida a solicitudes importantes, ni transmite cargas significativas al terreno.

Aunque se trata de una presa antigua se contaba con un conocimiento detallado de la geología de la cerrada y del vaso del embalse. En 1973 la Confederación Hidrográfica del Tajo redactó un proyecto de acondicionamiento en el que se estudiaron

Figura 2. Vista aérea de la presa al comienzo de las obras.

otras dos soluciones alternativas, consistentes en la ejecución de nuevas presas: una situada en el sitio de la existente y otra un poco aguas arriba, buscando una cerrada más estrecha. En dicho proyecto se realizó un estudio geológico y geotécnico de la cerrada muy completo, con sondeos, prospecciones eléctricas y ensayos de los materiales existentes en la zona.

En la geología de la cerrada de la presa de El Embocador se distinguen tres capas de materiales, con las características siguientes:

– La capa más somera está constituida por acarreos, gravas, arenas y limos, que forman subcapas de espesor variable y cierta continuidad. En superficie se encuentra tierra vegetal en las orillas, y sedimentos en el cauce y vaso del embalse, con espesores que varían entre 0,30 y 1,50 m; debajo hay un estrato de limos de potencia irregular que varía entre 0,50 y 6,50 m; y bajo los mismos un depósito de gravas y arenas también de potencia variable, que va de los 2 a los 10 m.
– Bajo los acarreos y depósitos modernos subyace una capa de arcillas o margas, también del Cuaternario, formada por solifluxión de las margas yesíferas del Mioceno que conforman los estribos del valle del Tajo. Se trata de una formación reciente, con alta impermeabilidad (en torno a 10^{-6} cm/s), bajo contenido en sulfatos (por debajo del 1%, ya que la mayor parte de las sales habrían sido lavadas durante el proceso de formación) y poca capacidad portante (no alcanza los 5 MPa). Tienen un espesor variable entre los 5,50 y los 8,50 m.
– Finalmente, en la base, encontramos el sustrato del valle, formado por materiales similares a los que aparecen en las laderas: yesos blancos mezclados con arcillas verdes grisáceas; que tiene una cierta consistencia puesto que no está alterado, pero que resulta de fácil disolución en el caso de que se produzca circulación de agua por el mismo.

3 PROCESO CONSTRUCTIVO

La planificación y el proceso constructivo de las obras estaban subordinados a la necesidad de mantener la presa en servicio y atender la campaña de riego de los canales, lo cual condicionaba indirectamente la solución técnica a adoptar para la ejecución de la impermeabilización del paramento y cimiento por aguas arriba.

La solución de ejecutar una pantalla frontal de impermeabilización con tecnología superjet grouting, aprovechando la accesibilidad al cuerpo del azud para utilizarlo como plataforma de trabajo, ha permitido satisfacer los requisitos de servicio existentes y contar con interesantes ventajas en varios aspectos del proceso constructivo, como la ejecución de un desvío del río más simple y la

reducción de los daños potenciales que se podrían producir en el área de trabajo en caso de desbordamiento por riada.

La ejecución de un blindaje frontal mediante un muro de hormigón armado, ejecutado con medios convencionales, hubiera obligado a dejar en seco el pie de aguas arriba del azud (mediante un sistema de ataguiado y de desvío del río de ejecución compleja). Sin embargo la construcción de la pantalla desde arriba solo precisa un vaciado parcial del vaso, lo cual se ha hecho aprovechando los canales de toma de la antigua central hidroeléctrica de margen derecha.

Las dos centrales existentes en la presa de El Embocador estaban abandonadas, encontrándose en estado de ruina la situada en la orilla izquierda (junto a la toma del canal de Las Aves), y habiendo sido derruida la de la derecha. De esta última se conservaban los canales de toma por los que se estableció el desvío del río. Se trata de dos canales rectangulares de 4 m de anchura y un calado de 3,10 m (desde la solera, en la embocadura, hasta el labio del vertedero del azud) con una capacidad de hasta 75 m^3/s. En uno de los canales se dispuso una compuerta de tablero de accionamiento manual, para tener un pequeño margen de regulación de cara a satisfacer lo mejor posible los compromisos de riego y caudal ecológico aguas abajo de la presa (que está establecido en 6 m^3/s a su paso por Aranjuez).

Una vez desviadas las aguas del Tajo se habilitó una pasarela de acceso a la parte superior del azud, la cual se realizó colocando un puente metálico sobre los citados canales de desvío. Seguidamente comenzó la retirada de la vegetación que se había desarrollado sobre el paramento del azud, y una vez efectuada la limpieza del mismo se extendió una capa de tierra compactada para nivelarlo y protegerlo del tránsito de la maquinaría.

Otra de las ventajas fundamentales de la solución adoptada es que se adapta mucho mejor que otras alternativas a la geología de la cerrada y vaso del embalse simplificando la ejecución y reduciendo los plazos y daños potenciales a afrontar durante las obras. La ejecución in situ de un muro frontal de impermeabilización exige, como se ha dicho, vaciar totalmente el embalse hasta dejar en seco el pie de aguas arriba de la presa, lo que hubiera requerido crear un recinto con ataguías que fuese prácticamente estanco. Ello, con la geología existente (con una capa subhorizontal de gravas y arenas muy permeables), hubiera obligado a sellar previamente, mediante algún elemento impermeabilizante, todo el perímetro del recinto situado entre la ataguía y la presa, y a contar a su vez con un significativo sistema de achique en el mismo; y habría precisado un desvío del río más potente, y de nueva ejecución en su totalidad, puesto que los daños que produciría en este caso un desbordamiento por riada serían mucho más graves.

La solución adoptada para impermeabilizar el paramento de aguas arriba aprovecha la forma de este tipo de azudes, que cuentan con un paramento de aguas abajo de gran anchura y poca pendiente; lo que permite, con un pequeño relleno de tierras sobre el mismo, habilitar la plataforma de trabajo de la maquinaria necesaria para ejecutar una pantalla tipo velo con tecnología superjet grouting. De esta forma no fue necesario crear un recinto estanco por aguas arriba, ni rebajar el nivel de embalse hasta pie de presa, ni potenciar el sistema de desvío del río. La pantalla se empotró unos 2 m en el horizonte de margas subyacente a la capa de gravas y arenas, alcanzando una profundidad media de unos 10,50 m (medida desde el labio de la presa).

Las instalaciones para la ejecución de la pantalla (silos de cemento, bomba y elementos de control de la inyección) se instalaron en el estribo derecho de la presa, de forma que solo fue preciso situar la máquina de perforación/inyección sobre el azud, con lo que el riesgo que se asumía en caso de un

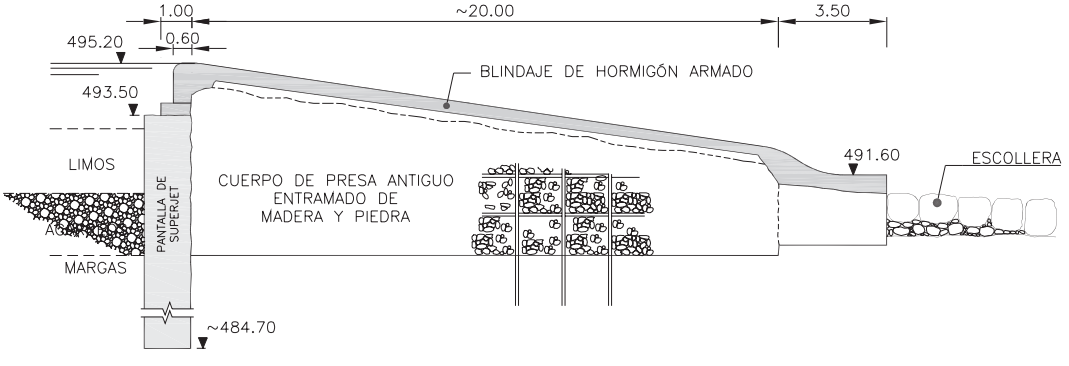


Figura 3. Sección tipo de la presa rehabilitada.

Figura 4. Vista aérea de la presa durante la construcción.

posible desbordamiento era muy bajo, puesto que la maquinaria, autopropulsada, podía retirarse en poco tiempo del mismo.

La parte superior de la pantalla, en la que se acumula el material de resurgencia que viene desplazado desde abajo por la inyección, se descabezó (unos 2 m) hasta alcanzar la zona en la que el material era más sano y homogéneo. La parte alta del paramento de aguas arriba quedo constituida por un muro que apoya sobre la pantalla de impermeabilización y le da continuidad. Se trata de un muro convencional de hormigón armado, de 0,60 m de espesor, rematado con un acuerdo circular que conforma el umbral del labio del vertedero de la presa. La ejecución de este acuerdo se cuidó especialmente para que estuviera perfectamente nivelado, puesto que el azud tiene una longitud muy grande y vierte una lámina de agua muy pequeña, siendo muy sensible a los defectos que se pueden producir en el mismo.

Una vez ejecutado el paramento de aguas arriba se efectuó el blindaje del resto del cuerpo del azud. Se preparó la superficie del mismo, picándose y saneándose las zonas deterioradas, y se ejecutó una losa de hormigón armado de 0,30 m de espesor. El extremo de aguas abajo se remató con un acuerdo de forma similar al existente al comienzo de las obras, y todo el pie de presa se ha protegido con una capa de escollera gruesa, formada por bloques de piedra caliza de al menos 1.000 kg de peso.

También, se adecuaron los canales utilizados para el desvío del río como desagües de fondo, de los que la presa carecía. Se restauraron los muros cajeros, se regularizó la solera del canal hasta su salida al cauce, y se colocaron compuertas de accionamiento motorizado: una compuerta de tablero en el canal izquierdo y una de tablero y una Taintor en el derecho, que es el que se empleará habitualmente para regular. Esta presa, aunque cuenta con un volumen de embalse muy pequeño, disponer de una pequeña capacidad de regulación es importante, pues el caudal de paso del Tajo por Aranjuez está fijado en un mínimo de 6 m^3/s por ley. Este caudal se suelta desde los embalses de cabecera de la cuenca, pero en el tránsito, de unos dos días de duración, se producen desajustes que tienen que ser corregidos. Esta función de reajuste se podrá llevar a cabo en la presa de El Embocador gracias a los desagües de fondo con que ha quedado dotada y a un innovador sistema de telemando (vía SAIH) que permite conocer los caudales servidos a los canales de riego, los vertidos sobre el azud y los evacuados por los desagües de fondo.

Finalmente, se acondicionó toda la zona del estribo derecho y el camino de acceso a la presa, quedando dos zonas diferenciadas: una en donde se ubican las instalaciones (vallada y con acceso limitado al personal de explotación) y otra acondicionada como mirador de acceso libre al público.

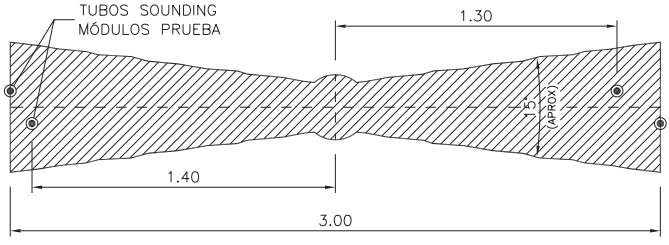
4 CARACTERÍSTICAS DE LA PANTALLA VELO DE SUPERJET

El jet grouting es una técnica de mejora del suelo que consiste en la inyección radial de un fluido a alta velocidad (caudal de 50 a 150 l/min), provocando la erosión y rotura del suelo y su sustitución y mezcla por el material inyectado. Al ir elevando el varillaje da lugar a columnas más o menos irregulares, de entre 0,6 y 2 m de diámetro, constituidas por el material inyectado que ocupa la zona central y una mezcla del mismo con parte del suelo existente en la periferia. El fluido empleado en la inyección es habitualmente lechada de cemento, por lo que las columnas formadas son (en función del suelo en el que se efectúe el tratamiento) de un hormigón mal dosificado si se está tratando un aluvial, o un suelo cemento en caso de materiales limosos o arcillosos.

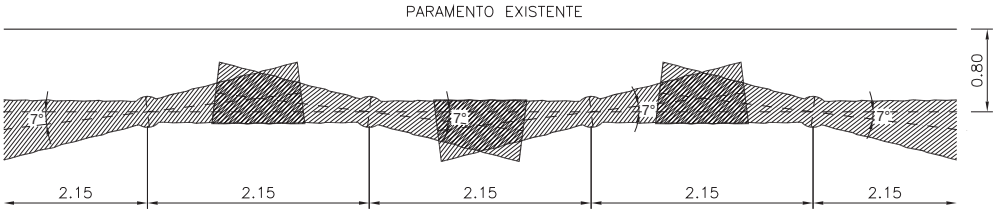
El superjet grouting es una evolución de los sistemas de jet de doble fluido (inyección de lechada envuelta por un cono de aire comprimido) en la que se utilizan herramientas de inyección más modernas y potentes (bombas, varillaje, toberas, etc.) para contar con una energía de inyección significativamente más grande (caudales de 300 a 450 l/min). Lo cual unido a que se emplean sistemas de monitorización del proceso más avanzados, que posibilitan controlar mejor la rotación del varillaje y la velocidad de extracción, permite la obtención de columnas de mucho mayor diámetro (entre 3 y 5 m) y la ejecución de otras geometrías tipo paneles o velo.

En la presa de El Embocador se optó por este tipo de inyección ya que ofrece un mayor control de la ejecución, pudiendo ejecutarse pantallas tipo velo. En este caso, en el que el tratamiento se realiza junto a una obra histórica, esto era fundamental. La inyección tenía que ser por un lado agresiva, para que fuera efectiva, se trataran bien los materiales del cimiento y se creara una pantalla suficientemente impermeable; y por otro lado debía ser muy cuidadosa con la estructura existente, que podía ser dañada con facilidad. Las técnicas convencionales de jet, si bien podían haber tratado el terreno adecuadamente formando una pantalla de pilotes secantes, habrían afectado al cuerpo de presa al introducir en el mismo los fluidos, inyectados a muy alta presión, pudiendo causar su desmoronamiento y el levantamiento de la losa superior de protección.

La construcción de una pantalla tipo velo soslaya estas dificultades. En el varillaje del superjet grouting se dispone de dos boquillas de inyección contrapuestas, por las que se introduce la lechada de cemento y el aire a presión. En el proceso de ejecución el varillaje no rota completamente sino que solo se mueve unos 15° alternativamente para



DETALLE UN MÓDULO DEL VELO



VELO DE SUPERJET GROUTING

Figura 5. Pantalla tipo velo ejecutada con superjet grouting.

un lado y otro, por lo que en vez de obtenerse una columna resulta un panel más o menos plano (con forma de lazo). Esto ha permitido crear en la presa de El Embocador una pantalla frontal de impermeabilización sin dañar el área frontal del azud, al realizarse la inyección en paralelo al eje de la presa y sin focalizarla en ningún momento contra el mismo.

Para la definición de los parámetros de inyección se realizaron tres columnas de prueba, dos en el estribo derecho y una en el izquierdo, determinándose los valores óptimos de la inyección: la presión de inyección, el caudal de mezcla inyectada y el ritmo de extracción de la lanza. En los módulos de prueba se verificó el alcance y la potencia del flujo mediante la excavación y examen de las columnas realizadas, así como con la técnica de los tubos sounding, en los que se comprueba hasta que punto la inyección es capaz de limpiar las muestras de pintura que se disponen sobre los mismos.

Las características de la pantalla ejecutada en la presa de El Embocador son las siguientes:

- Tipo: velo ejecutado con superjet grouting
- Ancho aproximado de los módulos: 3 m
- Distancia entre ejes de perforación: 2,15 m
- Diámetro de las toberas de inyección: 4,6 mm
- Densidad de la lechada: 1,4 ~ 1,5 t/m^3
- Presión de inyección: 300 bar
- Caudal de la mezcla inyectada: 380 ~ 390 l/min
- Ritmo de extracción de la lanza: 5 min/m
- Ángulo de rotación: 15° (zigzag)
- Presión de aire: 8 ~ 10 bar

5 CONCLUSIONES

Dentro de los tipos de jet grouting, existentes en la actualidad, el superjet puede conseguir un mayor alcance de tratamiento del terreno (diámetros de 3 a 5 m) al emplear maquinaría más potente e inyectar caudales mucho mayores. A su vez, el superjet grouting cuenta con elementos de monitorización más sofisticados que permiten dirigir la inyección de forma más precisa, por lo que puede utilizarse también en operaciones delicadas (en las que haya que salvaguardar estructuras antiguas para que no resulten dañadas por la agresividad de la operación).

Esta técnica se ha empleado de forma satisfactoria en la rehabilitación de la presa de El Embocador (estructura que data de principios del siglo XVI), en la que era necesario cortar las filtraciones a través del cuerpo de presa y el cimiento y a su vez era fundamental preservar íntegramente la fábrica original, de interés histórico. Adyacente al paramento de aguas arriba de la presa se ha ejecutado una pantalla de impermeabilización tipo velo (inyección en abanico de 15°) de 160 m de longitud y 10,50 m de profundidad, en el plazo de un mes. La rapidez de la ejecución, y la disposición de la plataforma de trabajo a nivel del vertedero, ha permitido trabajar con agua en el embalse, manteniendo el servicio y con un sistema de desvío del río muy simple.

El procedimiento empleado es una innovación importante dentro del campo de rehabilitación de presas, recomendable cuando se necesita construir pantallas de sellado de estas características. Como resumen general, las ventajas fundamentales de las pantallas de superjet tipo velo, son su seguridad, precisión y rapidez. Ello ha permitido que, en la rehabilitación de esta presa, se consiga:

- Trabajar desde una plataforma superior fuera del agua.
- Desvío del río más sencillo, más económico y con pocos elementos en riesgo.
- Mantenimiento del embalse en servicio mientras se ejecutaban las obras.
- Reducción del plazo de ejecución de las obras.
- Inyección por difusión de las oquedades del paramento frontal de la presa, sin dañar la estructura de ésta.

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Proyecto de adecuación de la presa del Torrollón al Reglamento técnico sobre seguridad de presas y embalses

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ABSTRACT: In December, 2003, in the framework of the First Review and Analysis of Security of the dam, several deficiencies were detected in *El Torrollón* dam (Huesca, Spain) concerning with the safety of the dam. Principally a progressive increase in the filtrations and movements was detected in the concrete dike in which are located the dam outlet works, placed in the left stirrup. For all this, the Project of Adequacy of *El Torrollón* dam to the Technical Regulation of Security of Dams and Reservoirs has been required by the *Ebro Basin Authority*. The main measures of this project are the improvement of the stability of the left stirrup of the dam by means of the construction of a mass concrete wall leaned downstream, the realization of a new lateral spillway; and the construction of new outlets, with a tower in their mouth and conduits in a false tunnel of more than 220 m length.

1 INTRODUCCIÓN Y ANTECEDENTES

El embalse del Torrollón se sitúa en el barranco Común (o Grande) en el término municipal de Lalueza (provincia de Huesca, España) y tiene como principal finalidad la de servir de elemento regulador del canal del Flumen durante la temporada de riegos, para lo cual dispone de una capacidad de 1,79 hm^3.

La presa, cuya titularidad recae sobre la Confederación Hidrográfica del Ebro, está dividida en dos partes claramente diferenciadas:

– Un dique principal de materiales sueltos de 1115 m de longitud de coronación y 29.20 m de altura. Está constituido por un material tipo todo uno con una serie de drenes en la zona de aguas arriba y un dren chimenea vertical, con taludes 2.5H/1V.
– Un dique de hormigón en masa de 31.70 m de longitud y 20.23 m de altura donde se sitúan todos los órganos de desagüe de la presa: aliviadero, toma y desagüe de fondo. La obra es de geometría variable, predominando los paramentos verticales a excepción de la zona donde se ubica el aliviadero. En su interior existe una cámara para el alojamiento de las válvulas del desagüe de fondo y toma. Este dique, por situarse en la margen izquierda se ha denominado tradicionalmente "estribo izquierdo", aunque realmente debe considerarse a todos los efectos como un dique independiente de la presa de materiales sueltos.

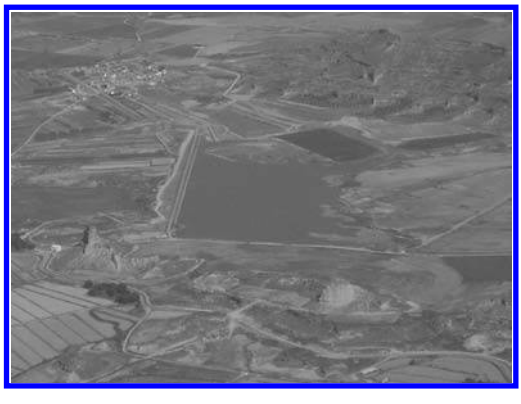

Figura 1. Vista aérea del embalse.

Ambos diques están alineados en planta y sólo distan entre sí unos 70 m merced a un pequeño afloramiento del terreno que dificulta apreciar a simple vista la discontinuidad existente, siendo sencillo confundir el dique de materiales sueltos con el terraplén de la carretera que discurre por coronación debido a que los materiales son muy similares. En realidad, el dique de hormigón ("estribo izquierdo" a partir de ahora para seguir la denominación tradicional) se sitúa como cierre de

Figura 2. Vista del estribo izquierdo de la presa desde aguas abajo.

un desmonte específicamente abierto en el terreno para alojar el canal de descarga del aliviadero y el canal del Flumen.

Las obras de construcción de la presa concluyeron el 1 de diciembre de 1983. Tras la entrada en servicio de la presa y especialmente entre el periodo comprendido entre 1985 y 1988 se pusieron de manifiesto una serie de daños en forma de agrietamientos que afectaban seriamente al hormigón del estribo izquierdo, así como al canal de descarga del aliviadero y los primeros 150 metros de la prolongación del canal del Flumen aguas abajo de la presa. Con vistas a subsanar los daños fue redactado, en abril de 1988, un Proyecto de Reparación. Este proyecto señalaba, como causa principal de los daños, la existencia de asientos en el dique de hormigón, debido a un mecanismo de erosión interna en una capa de arenisca muy porosa que existe en la cimentación.

Esta reparación es el primer antecedente del largo historial conflictivo que posee la zona del estribo izquierdo y que ha dado lugar a la ejecución de diversas obras correctivas como pantallas de impermeabilización, inyecciones de baja presión, sellado de juntas, anclajes de estabilización y realización de drenes.

2 PRIMERA REVISIÓN Y ANÁLISIS DE SEGURIDAD DE LA PRESA

En diciembre de 2003, en el marco de la *1ª Revisión y Análisis de Seguridad de las presas*, realizada bajo la supervisión y coordinación de la Confederación Hidrográfica del Ebro, fueron detectadas una serie de deficiencias que volvían a poner en cuestión el grado de seguridad de la presa y que hacía pensar en una reproducción de los antiguos problemas. Principalmente fue detectado un progresivo aumento de las filtraciones y movimientos en el estribo izquierdo.

También ha sido definitorio contrastar los asientos que se producen en coronación en diferentes campañas a lo largo de la vida de la presa. En la Figura 5 se representa la comparación entre una campaña realizada en febrero de 1986 con otra campaña realizada en octubre de 2003. Tal y como puede apreciarse frente al asentamiento relativamente normal del centro y derecha del gráfico (correspondiente al dique de materiales sueltos) se

Figura 3. Ejemplo de evolución de movimientos en una junta del estribo izquierdo.

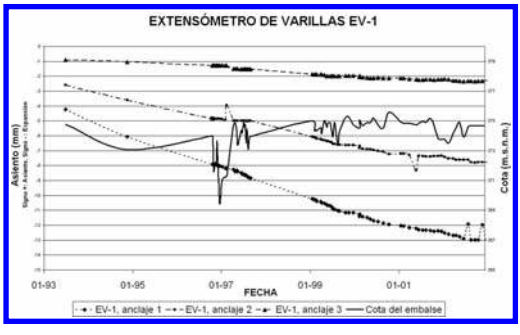

Figura 4. Ejemplo de evolución de expansiones en extensómetro de varillas en el estribo izquierdo.

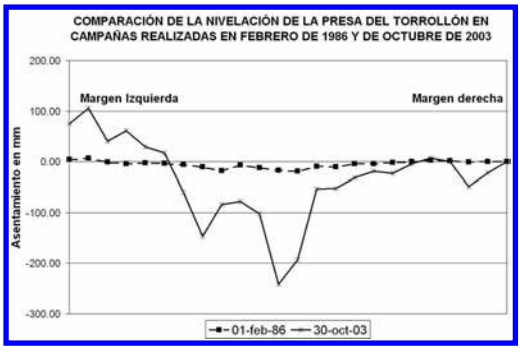

Figura 5. Comparación de la nivelación de la presa del Torrollón en campañas realizadas en febrero de 1986 y de octubre de 2003.

da una elevación anormal en la izquierda, que corresponde al dique de hormigón.

En dicha revisión, a partir de la información contenida en el Archivo Técnico, de la inspección directa de la obra y de los cálculos y comprobaciones realizados, se concluyó que había una falta de adecuación al Reglamento Técnico de Seguridad de Presas y Embalses (referente normativo español de la mayoría de las presas en explotación) en lo que concernía a la seguridad de la presa.

En concreto, desde el punto de vista de la seguridad estructural, fueron detectadas las siguientes circunstancias que evidenciaban existían claros síntomas de inseguridad:

- La pantalla que envuelve perimetralmente el pie de aguas arriba del estribo izquierdo mostraba indicios, debido al aumento de las filtraciones, de haber perdido su eficacia, lo que hacía temer un posible proceso de degradación y erosión interna en la capa de areniscas pobremente cementadas donde apoya la sección de mayor altura del estribo. Los procesos de arrastre de arena y finos podían estar conduciendo a una aceleración de los movimientos de giro (vuelco) del muro-estribo y de la apertura de sus juntas de hormigonado, especialmente las más bajas, reproduciéndose de forma más o menos fiel el proceso de daños que tuvo lugar durante la primera puesta en carga de la presa.
- La estabilidad del estribo izquierdo no estaba garantizada, al no darse las condiciones incluidas en los cálculos de estabilidad del proyecto de reparación de 1988 ya que:
  1. La subpresión detectada con los piezómetros era mayor que la asumida en los cálculos del proyecto.
  2. Los anclajes que se instalaron en el estribo izquierdo podían haber dejado de ser funcionales al haberse superado ampliamente las 100 t previstas (en mayo de 1992 se sobrepasó el rango máximo, cifrado en 200 t, de la célula de carga que controlaba uno de los anclajes).
  3. En el caso más desfavorable en que ya no se pudiera contar con la contribución de la fuerza de los anclajes se tendría en la sección más desfavorable unos coeficientes de seguridad reducidos, que serían según el proyecto de Reparación de 1988 iguales a 1.28 frente al deslizamiento y 1.13 frente al vuelco. Se destaca que estos coeficientes de seguridad se obtenían suponiendo una subpresión en el pie de aguas arriba de la sección más desfavorable igual al 80% de la hidrostática, la cual se podía estar superando según recogían los valores registrados en los piezómetros.

Ante esta situación de evolución desfavorable de los movimientos y filtraciones, que era confirmada por todos los sensores instalados (piezómetros, extensómetros de varillas, clinómetros, aforadores, hitos en coronación y medidores de apertura de juntas) las principales conclusiones fueron las siguientes:

- Asumir la deficiencia de los modelos de comportamiento empleados hasta entonces, puesto que la evolución del estribo izquierdo había estado caracterizado por un progresivo aumento de las filtraciones y los movimientos sin que hubiera saltado previamente la alarma.
- Llamar la atención sobre el desconocimiento del estado real de la cimentación del estribo izquierdo, especialmente de la capa de areniscas muy porosas que podían estar sufriendo un proceso evolutivo de erosión interna.
- Realizar de manera urgente algún tipo de medida correctora, lo que se tradujo en descender el nivel de embalse y mantener desde entonces un límite superior para el mismo compatible con la seguridad de la presa (el empuje hidrostático medio a lo largo de estos años ha sido sólo de un 40% respecto el máximo normal).

Adicionalmente a estos problemas la Primera Revisión puso de manifiesto otras carencias de segundo orden como deficiencias en el paramento de aguas abajo del dique de materiales sueltos, debido a una importante erosión superficial producto de la escorrentía, y deficiencias en la seguridad de las instalaciones y en los accesos a la presa.

3 CARACTERIZACIÓN DEL PROBLEMA

Para la comprensión del problema hay que tener en cuenta que, en primer lugar, el dique de hormigón, carece de las juntas de hormigonado que deberían ser preceptivas en la ejecución correcta de obras de estas características, de modo que, en lugar de un dique formado por bloques claramente delimitados de cimentación a coronación, existe una compleja disposición de tongadas de diferentes tamaños formando numerosos bloques independientes.

Sin duda detrás de esta forma de ejecución están gran parte de los problemas de filtraciones a través del cuerpo del dique que se han tratado de resolver, sin éxito, a lo largo de la historia de la presa mediante sucesivas inyecciones y cosido de los bloques entre sí.

Por otra parte, la presa descansa sobre una cimentación fuertemente escalonada, con una alternancia de areniscas, margas arcillosas y limolitas. Estas capas, de espesor que oscila entre varios decímetros y algunos metros, están dispuestas de forma sensiblemente horizontal y sin deformaciones tectónicas reseñables.

El yacente inmediato al pie del estribo en su zona de mayor altura está constituido por una secuencia de cinco diferentes paquetes de areniscas y margas o limolitas, cuya secuencia desde la superficie es la siguiente:

- Limolitas muy arenosas que lateralmente pasan a arcillas margosas o margas arcillosas. Su espesor oscila entre los 2.0 y 4.0 m, aumentando desde margen izquierda hacia margen derecha. Originalmente, el canal de descarga del aliviadero y la toma del Canal del Flumen se apoyaban sobre los términos basales de este horizonte.
- Arenisca de grano medio a fino y muy porosa, con una matriz de naturaleza limo-arcillosa y escasa o nula presencia de cemento. Su espesor oscila entre los 3.0 y 4.0 m. Actúa como nivel de cimentación inferior del dique de hormigón y resulta especialmente problemático debido a que su porosidad (estimada en torno al 30%) permite que se comporte como un nivel acuífero con fácil circulación del agua.
- Margas arcillosas marrones con pasadas amarillentas y granates, duras y de plasticidad media. Su espesor oscila entre 2.0 y 3.0 m.
- Alternancia decimétrica de niveles de areniscas arcillosas de grano fino y fuertemente cementadas y margas arcillo-arenosas. Su espesor total oscila entre 1.0 y 2.0 m.
- Margas arcillosas marrones, de plasticidad media-alta y consistencia dura, con ocasionales pasadas de areniscas grises de grano fino y muy cementadas. Su espesor es, al menos, de 4.0 m.

La combinación de ambas circunstancias, una cimentación escalonada con diferentes materiales (uno de los cuales es además proclive a la erosión interna) y un dique con bloques dispuestos de

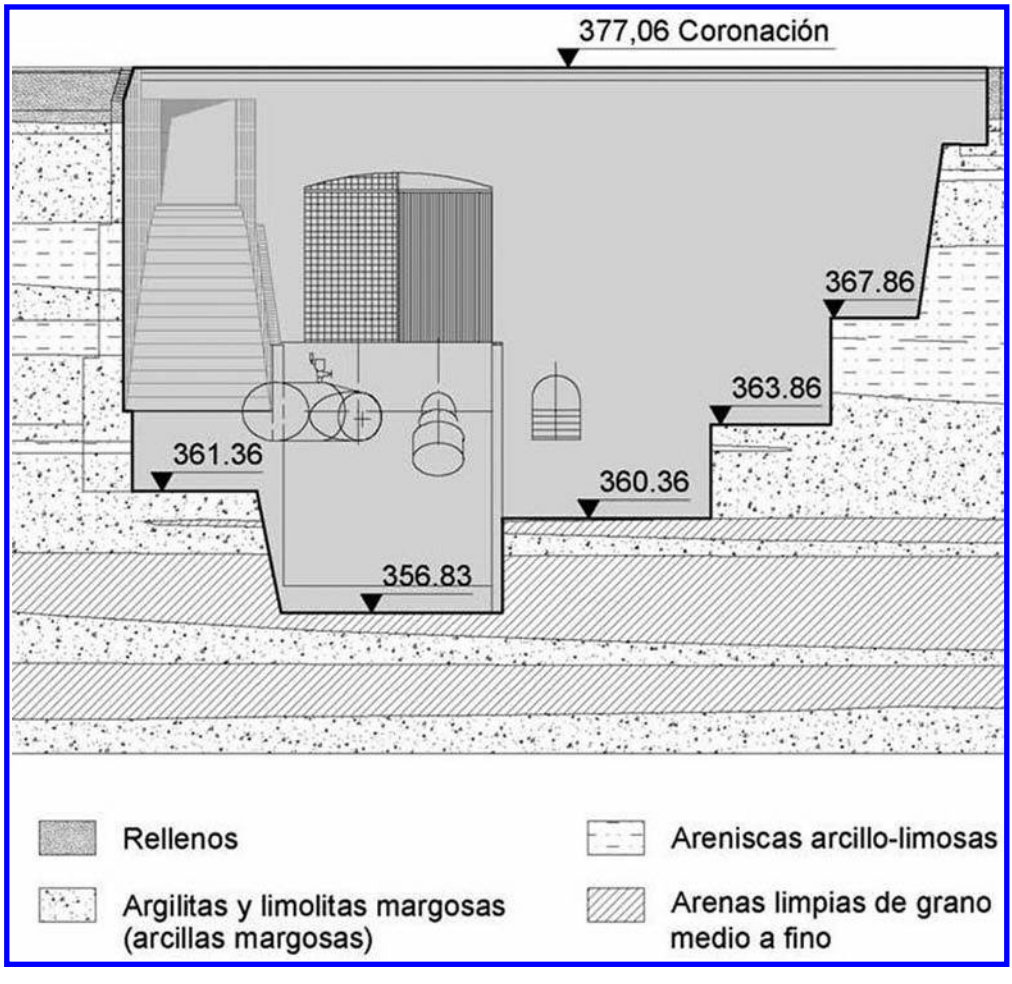

Figura 6. Detalle de las capas de la cimentación existente en el estribo izquierdo de la presa.

manera poco acertada, parecen ser la explicación de toda la problemática sufrida en la zona del estribo izquierdo.

Así, el mecanismo que se propone para explicar el problema sería achacable, una vez que los ensayos realizados han descartado posibles fenómenos expansivos en el cimiento, a una erosión interna prolongada en la capa de areniscas permeables. Esta erosión a su vez genera, debido a la inapropiada configuración de los bloques del dique y a los distintos materiales sobre los que se apoya en cimentación, una serie de movimientos diferenciales en diversas zonas del estribo que no pueden ser absorbidos por las juntas, originando así nuevas grietas y aumento de las filtraciones.

4 SOLUCIÓN ADOPTADA

La Primera Revisión del año 2003 consideraba imprescindible acometer medidas para reducir el nivel de riesgo, medidas que habrían de centrarse necesariamente en reformar la zona del estribo izquierdo, si bien se podía aprovechar para acometer otras carencias que pudieran estar para entonces aún pendientes, como la adecuación de los accesos, instalaciones y paramentos en la presa de materiales sueltos.

En lo que respecta al dique de hormigón o estribo izquierdo, la solución adoptada, con vistas a detener la progresión de movimientos y filtraciones existentes, ha sido dotar de mayor inercia al estribo, rellenando los huecos existentes (como la cámara de válvulas) y adosando aguas abajo un elemento de refuerzo que aumente la inercia del conjunto. Con esta ampliación de la base del estribo se pretende además reducir el gradiente hidráulico y la velocidad del agua disminuyendo el riesgo de erosión interna. Adicionalmente, se consideró conveniente tratar el terreno de cimentación para mejorar su resistencia e impermeabilidad.

La ejecución de esta solución implica poner fuera de servicio los actuales órganos de desagüe de la presa. Esto conlleva a un traslado respecto de su situación actual tanto al aliviadero, a la toma como al desagüe de fondo lo que implica un nuevo aliviadero y una nueva conducción para la toma y el del desagüe de fondo.

En la elección de la solución se ha tenido muy en cuenta que en el año 1988 se proyectó y ejecutó una gran reparación que no obtuvo los resultados apetecidos, justificando con ello la decisión que las obras a desarrollar por el nuevo proyecto no tenían que ser concebidas como una simple reparación "tradicional", esto es, aplicando las clásicas medidas correctivas como nuevas inyecciones, anclajes o pantallas, sino que era más prudente plantear una solución que, sin descartar la aplicación adicional de las medidas citadas, garantizara que en caso de volverse a producir signos de deterioro en la zona del estribo izquierdo la repercusión sobre la seguridad de la presa fuera menor que la existente en la actualidad (los órganos de desagüe de la presa se ubican precisamente en dicho estribo).

Otro condicionante adicional estribó en que los responsables de la explotación de la presa plantearon la necesidad de que la ejecución de las obras a proyectar no implicase modificación práctica de las condiciones de servicio del Canal del Flumen, por lo que algunas de las actuaciones que debían proyectarse habrían de poderse llevar a cabo exclusivamente en el periodo de parada de

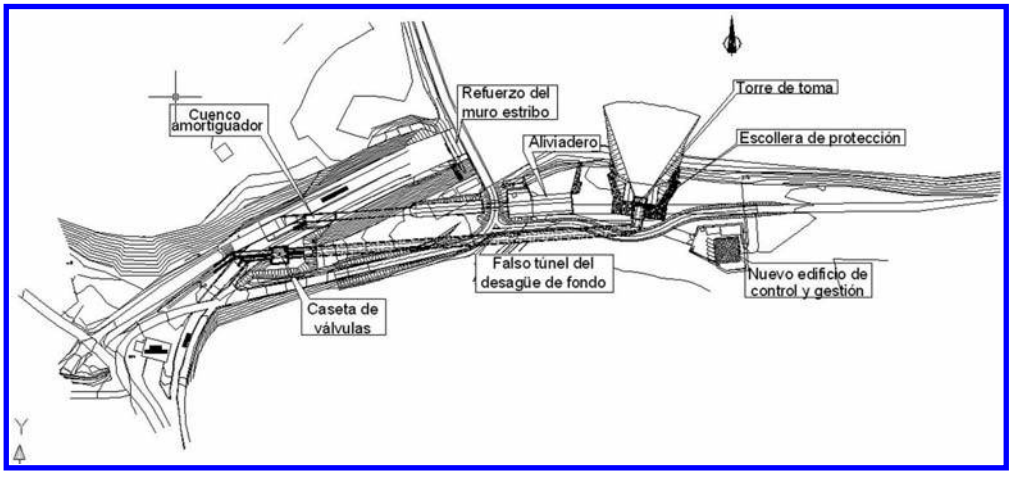

Figura 7. Planta general con las principales actuaciones recogidas en proyecto.

riegos anual, de cinco meses de duración máxima. Fuera de dicho periodo, la presa debería seguir prestando su servicio.

4.1 *Refuerzo del estribo*

Fueron tanteadas las ventajas y los inconvenientes de dos posibles soluciones, ejecutar un terraplén aguas abajo y adosar un muro de hormigón en masa.

En el caso del terraplén, las rigideces de éste y del muro de hormigón son distintas, y aunque desde el primer momento ejercerá un cierto empuje horizontal de sentido contrario al del agua en el muro, no llegará a trabajar de forma efectiva, pues el rango de deformación compatible de ambos es distinto. Adicionalmente, probablemente habría que adicionar un cierto refuerzo en el lado aguas arriba para el caso de embalse vacío, no solo por inestabilidad sino porque las grietas existentes en el muro empeorarían con los nuevos esfuerzos que le provocarían las tierras. Por otra parte, la continuidad de las filtraciones provocaría el lento deterioro del muro y además sería necesario disponer drenes en el terraplén.

El muro de hormigón en masa presenta la ventaja de que tendrá una rigidez similar a la del existente, con los huecos rellenos de hormigón, y, si queda bien adosado, trabajarán de forma conjunta aumentando la inercia del conjunto y reduciendo las filtraciones.

En conclusión, se adoptó como solución la construcción de un refuerzo en el estribo izquierdo, realizando las siguientes medidas:

- Construcción de un muro de hormigón en masa adosándolo aguas abajo del estribo actual. Este muro tendría una anchura en coronación de 6 m, situándose 3.2 m por debajo de la coronación del dique actual y con un paramento visto de talud 1H:5V.
- Macizado de la estructura existente mediante el hormigonado de los huecos que restan inercia al conjunto: cámara de válvulas y galería de acceso, aliviadero y tuberías. Previamente se vaciarán los elementos electromecánicos existentes, lo que requiere que estén operativos los nuevos órganos de desagüe.
- Inyecciones de sellado de fisuras en el estribo actual.
- Inyecciones de consolidación e impermeabilización del terreno de apoyo del muro de refuerzo, con la posibilidad de estudiar una pantalla continua de bentonita-cemento en esa zona.
- Construcción de zanja- dren de captación de filtraciones en el pie del muro de refuerzo.

4.2 *Nuevo aliviadero*

El aliviadero actual se sitúa sobre el dique de hormigón de la presa, y si este era reparado por motivos de estabilidad, parecía lógico el traslado de la ubicación del aliviadero fuera de la zona conflictiva.

Se analizó si la ubicación más conveniente para el nuevo aliviadero era a la izquierda o derecha de su emplazamiento actual. En ambos casos se observó la posibilidad de cimentar el futuro aliviadero en un estrato margoso de más de 6 m de potencia, salvando de esta manera las capas conflictivas de areniscas y manteniendo la cota del labio o umbral en el mismo nivel que el aliviadero existente.

La ubicación en la margen derecha del estribo aprovecharía la zona de terreno que separa el dique de hormigón del dique de materiales sueltos, de manera que haciendo un pequeño corte en ella se tendría espacio para poder encajar el aliviadero, con un vertedero frontal y un canal de descarga que llegaría hasta el canal de descarga actual. Esta solución tenía la ventaja de no interferir el canal del Flumen.

Sin embargo, existían otros inconvenientes que hicieron descartar la solución. Para empezar la configuración del terreno obligaría a la realización de un canal con una pendiente uniforme asociado a un gran volumen de excavación o bien la realización de una rápida de gran pendiente al final para salvar el desnivel existente. Además tiene el inconveniente de que el canal de descarga tendría mucha longitud para alejarlo lo suficiente del estribo y así evitar

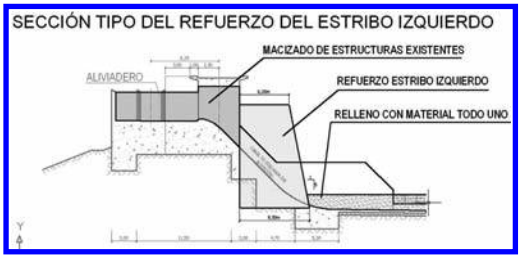

Figura 8. Sección tipo del refuerzo del estribo izquierdo.

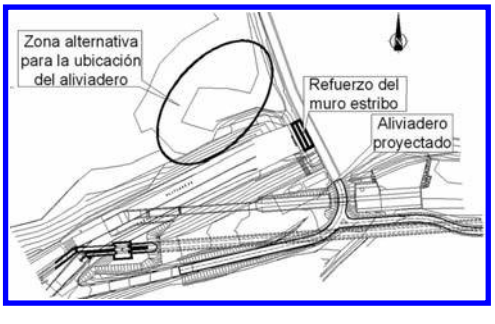

Figura 9. Ubicación alternativa para el aliviadero.

afectar a la estabilidad del dique de hormigón. Por último la presunta ventaja para el aliviadero de no tener que cruzar el canal del Flumen, se convertía en inconveniente para el desagüe de fondo, que entonces debería cruzar el canal de descarga del aliviadero, además de requerir excavaciones de gran profundidad.

La segunda posibilidad, descartando el mantenimiento del aliviadero en la situación actual y evitando la interferencia con las actuaciones de estabilidad del estribo, era colocar el aliviadero en la margen izquierda de éste, conformando una tipología de vertedero lateral, con un cuenco de recogida y posteriormente un canal cruzando bajo el camino de coronación de la presa, llevando el canal de descarga hasta prácticamente la proximidad de la central hidroeléctrica existente. El terreno en esta margen tiene una pendiente más suave que en la margen derecha del estribo, con lo que hay un volumen menor de excavación.

En contra esta solución se presentaba el riesgo de afectar a la estabilidad de la ladera, donde se localiza un esbelto y espectacular cerro testigo formado por areniscas y margas que semeja una gran torre (de ahí la denominación de "Torrollón" de la presa) cuyo valor medioambiental es notable. Esta posibilidad se evitó encajando una solución en el estrecho pasillo delimitado por esta forma geológica y el desmonte donde se ubica el canal del Flumen, de manera que los taludes de excavación no llegaran a afectar a la zona de apoyo basal del cerro. El canal de descarga termina en un cuenco amortiguador donde se provoca el resalto antes de incorporar el flujo al canal de descarga existente.

Habiendo optado por la solución de ubicar el nuevo aliviadero por margen izquierda y manteniéndolo lo suficientemente alejado del dique de hormigón para no afectar a su estabilidad, se consideró si su trazado se podría acortar llevándolo lo más cerca posible del pie de presa, pero finalmente se optó por la opción que funcionaba mejor desde el punto de vista hidráulico y que implica una longitud total de aliviadero de casi 199.58 m (incluyendo vertedero y cuenco amortiguador) para un caudal total de 22 m^3/s.

Figura 10. *Ladera* donde está previsto situar el canal de descarga del aliviadero proyectado. Al fondo a la izquierda puede observarse la *forma* geológica conocida localmente como "Torrollón" que da nombre a la presa.

4.3 *Nueva conducción de desagüe de fondo y toma*

Se ha asumido mantener la finalidad actual de los dos conductos, esto es, una conducción destinada al vertido directo en el canal del Flumen, y la otra para suministro a la central hidroeléctrica de pie de presa, existiendo la opción de combinar ambos por medio de una derivación.

Se barajaron inicialmente varias opciones: mantener las conducciones en la situación actual, o trasladarlas, al igual que se ha expuesto con el aliviadero, a la derecha o a la izquierda del dique de hormigón.

Mantener el emplazamiento en la situación actual, independientemente de las actuaciones de refuerzo del estribo, implica una gran incertidumbre en cuanto al éxito o no de la mejora debido a la experiencia que se tiene del anterior proyecto de reparación, ya que todo el conducto estaría cimentado sobre la capa de arenisca problemática.

Todo ello lleva a adoptar como solución desplazar el desagüe de fondo y toma próximos al emplazamiento del nuevo aliviadero, concretamente a la izquierda del mismo para que los conductos no se crucen con el canal de éste a la hora de enlazar con el canal del Flumen. De esta forma los ejes del aliviadero y del desagüe de fondo siguen un trazado en su mayor parte paralelo que distan 21.40 m entre sí.

La embocadura de los desagües irá adosada a una torre de toma de 11.50 m de profundidad que alojará la cámara de compuertas de seguridad de los dos conductos, mientras que las compuertas de regulación se sitúan en otra cámara situada al final de la conducción.

Al ser necesario una profunda excavación para instalar los conductos se plantearon dos posibles soluciones: dejar abierto el desmonte de la excavación o realizar una galería que alojara los conductos del desagüe de fondo. Debido a que sería necesario para mantener abierta permanentemente la excavación unos taludes muy tendidos que afectaría al cerro testigo se ha optado por la solución de galería.

Esta última solución presenta a su vez dos opciones posibles: 1) realización de un túnel mediante excavación tradicional o una tuneladora, y 2) abertura en trinchera para realizar un falso túnel rellenando posteriormente la excavación realizada.

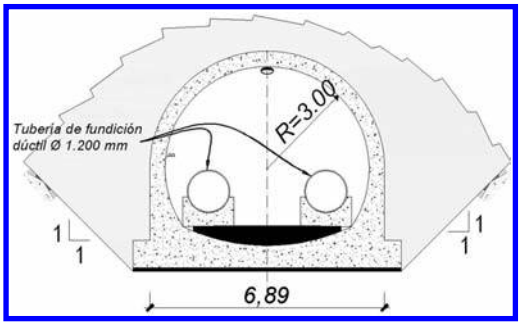

Figura 11. Sección tipo del falso túnel.

Eliminada la primera opción por su mayor coste se optó por la realización de un falso túnel de 222.58 m compuesto por dos arcos de circunferencia, de 3 m de radio para la bóveda superior y 5.71 m de radio para la contrabóveda inferior, que alojarán los dos conductos de fundición de 1200 mm de diámetro.

La longitud total del conducto que desagua en el canal del Flumen (desagüe de fondo propiamente dicho) es de 267 m mientras que la de la toma de la central hidroeléctrica es de 328 m.

5 CONCLUSIONES

Las obras proyectadas tratan de corregir los problemas actualmente existentes debido, por una parte, a una cimentación fuertemente escalonada en un tramo de escasa longitud y apoyándose en terrenos de diferentes características mecánicas, con una capa inferior proclive a la erosión interna y, por otra, a la ejecución del dique mediante una disposición poco ortodoxa de bloques y juntas. Estas circunstancias en conjunto facilitan los agrietamientos y movimientos diferenciales transmitidos desde la zona central donde los asientos son originados al apoyar en la capa de areniscas poco cementadas.

La solución adoptada implica asumir un diseño conservador, descartando volver a aplicar medidas correctivas clásicas de estabilización como nuevas inyecciones, anclajes o pantallas que ya tuvieron la oportunidad de ponerse en práctica a lo largo de más de 20 años sin que hayan surtido efecto. Con la solución planteada no sólo se aumenta la estabilidad del dique, sino que se garantiza que, en caso de reproducirse los mismos problemas, la repercusión sobre la seguridad de la presa será menor que la existente en la actualidad, puesto que se independiza totalmente el dique de hormigón de los órganos de desagüe de la presa.

Toda esta experiencia pone de relieve la utilidad de las revisiones periódicas para poner de manifiesto situaciones de riesgo, puesto que el dique de hormigón estaba experimentando en los últimos años un proceso alarmantemente evolutivo en sus movimientos y filtraciones.

En la actualidad la presa lleva 5 años con el nivel de embalse limitado aproximadamente un 65% por debajo del máximo, lo que ha supuesto una merma de su potencial hidroeléctrico y regulador, si bien en ningún momento el servicio a la zona regable del canal del Flumen se ha visto afectado por ello.

Esto, más el coste de las obras del proyecto, vuelven a hacer válida la vieja máxima de "lo barato es caro", por lo que se vuelve a insistir en la necesidad de prestar toda la atención durante la fase de proyecto a los estudios del terreno y, durante la fase de construcción, a vigilar intensamente la ejecución de la obras.

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Terraplén de contención de las laderas del embalse de Arenós en Puebla de Arenoso (Castellón)

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RESUMEN: Puebla de Arenoso se sitúa en la cola del embalse de Arenós, sobre una lengua de material de derrubio que constituye el pie de un deslizamiento cuaternario. En el estado actual, la ladera sobre la que se asienta se encuentra en equilibrio muy precario. Por dicho motivo se ha proyectado una obra de estabilización que consiste en un canal de hormigón que sostiene el pie de unos rellenos que alcanzan la cota 600 (40 m por encima de la cota del cauce).

Con estas medidas se consigue un aumento del coeficiente de seguridad frente al deslizamiento del orden de 40%, respecto a la situación primitiva. El diseño hidráulico ha condicionado en gran manera la solución adoptada, y la realización de un modelo a escala reducida ha permitido mejorar algunas características del diseño.

Los movimientos de la ladera están siendo controlados mediante un sistema de monitorización a base de inclinómetros y estaciones GPS, que sirven de apoyo a la obra en ejecución.

ABSTRACT: Puebla de Arenoso is located at the tail end of the Arenos reservoir, on a tongue of drift material which constitutes the toe of a quaternary landslip. In its present state, the slope on which it sits is in very precarious equilibrium. For this reason stabilization work was planned. This consists of a concrete canal that supports the toe of filling which reaches the 600 m level (40 m above the level of the river).

Compared to the original situation, these measures have achieved a 40% increase in the safety factor against slip. The hydraulic design has to a great extend determined the solution adopted, and the use of a scaled model has allowed for improvements in some of the design characteristics.

Movement of the slope is monitored by a system based on inclinometers and GPS stations, which serve to back up the work being carried out.

1 ANTECEDENTES

Históricamente la inestabilidad de las laderas del valle del río Mijares en su curso alto ha sido un fenómeno frecuente. Destacan por su magnitud los deslizamientos existentes próximos a Puebla de Arenoso, en donde se movilizaron millones de metros cúbicos en dos antiguos deslizamientos (paleodeslizamientos) de grandes dimensiones.

Estos dos grandes deslizamientos afectan principalmente a la margen izquierda, aunque en ambos casos quedan retazos de los mismos en la margen derecha. El primero de los deslizamientos se sitúa aguas arriba de Puebla de Arenoso, mientras que el segundo y mayor se sitúa frente al pueblo y aguas abajo del mismo.

Ambos deslizamientos antiguos han sufrido con posterioridad nuevos deslizamientos de menor entidad, afectando parcialmente a terrenos ya removidos. Hay que hacer especial referencia al gran número de deslizamientos habidos durante el final del año 1898 y comienzos del año 1990, período de intensa pluviometría.

Tras medio siglo de estudios, en el año 2005 el Ministerio de Medio Ambiente, a través de la Confederación Hidrográfica del Júcar (CHJ) consideró conveniente iniciar una obra de emergencia que paliara la antigua problemática de la estabilidad de Puebla de Arenoso. Después de considerar cuatro posibles soluciones de actuación en el río (marcos, bóvedas, presa y canal) se desecharon las dos primeras por temor al mal comportamiento de conductos

Figura 1. Vista general del río Mijares a su paso por Puebla de Arenoso, antes del inicio de las obras.

enterrados y la tercera por su elevado coste, y se optó por la solución en canal, encauzar el río en la zona del estrecho y rellenar con tierras los trasdoses de los muros del encauzamiento y además, restituir con terrenos compactados las superficies de la ladera. Se conseguía de esta forma restituir las condiciones topográficas originales, pero en esta ocasión con un cauce que no fuera susceptible de erosionarse.

2 DISEÑO GEOTÉCNICO

2.1 *Estudios previos*

La estabilidad de las laderas del embalse de Arenós ha sido motivo de preocupación desde los primeros estudios realizados por el Servicio Geológico de Obras Públicas para el proyecto del embalse (SGOP, 1966).

La localidad de Puebla de Arenoso domina una ladera, situada en la margen derecha, que se eleva aproximadamente 70 metros sobre el río Mijares (la cota aproximada de la población es la 630, mientras que el río discurre, aproximadamente, por la 560). En su sección más empinada esta ladera tiene un ángulo medio de 36º respecto a la horizontal.

En el año 1972 se lleva a cabo la primera investigación para estudiar su estabilidad (García Yagüe, SGOP, 1972). Este estudio sitúa geológicamente a Puebla de Arenoso sobre una lengua de material de derrubio que constituye el pie de un gran deslizamiento cuaternario ocurrido en la margen izquierda del río Mijares (ladera opuesta a Puebla) El material de deslizamiento se compone de grandes bloques de naturaleza caliza caóticamente distribuidos, inmersos en una matriz areno-arcillosa.

Por debajo del deslizamiento se sitúan las margas y calizas cretácicas del Aptiense, que entran en contacto, aguas debajo de la Puebla de Arenoso, con la facies Weald mediante una falla.

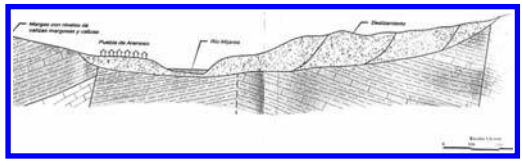

Figura 2. (García Yagüe, 1972). Croquis del perfil del deslizamiento sobre el que se sitúa Puebla de Arenoso.

Los estudios más recientes realizados por el CEDEX (Estaire et al., 2005 a y b) matizan ligeramente la geología descrita por el SGOP, sobre todo en la edad y profundidad de los deslizamientos (son varios los cartografiados). Otra aportación crucial de este estudio es la detección de unos niveles de material brechificado, de espesor decimétrico a métrico, en las proximidades del contacto entre los derrubios del deslizamiento y el sustrato cretácico. Estos niveles están constituidos por fragmentos angulosos de caliza margosa, de tamaño milimétrico a centimétrico, englobados en una matriz margo-arcillosa de color gris negruzco.

Este estudio del CEDEX incluía también la instalación y seguimiento de inclinómetros. Estos detectaron movimientos en la zona próxima al contacto entre los derrubios y el sustrato, siguiendo la línea de máxima pendiente de la ladera.

Finalmente, a la vista de los perfiles y las lecturas inclinométricas, el informe concluía que "la ladera se encuentra en una situación de equilibrio muy precario que produce movimientos de reajuste en la masa de derrubios, a lo que probablemente haya contribuido los sucesivos aumentos y descensos del embalse".

2.2 *Estudio de la estabilidad en su estado original*

Si bien la disposición de capas en el perfil es conocida, la naturaleza de los niveles que lo forman hace muy complicado ensayar sus características. En general, distinguimos tres niveles en el perfil.

- Un sustrato compuesto por las calizas y margas del Aptiense, que podríamos considerar como de gran resistencia. En este nivel las resistencias a compresión simple en calizas y margas oscilaron entre 6 y 56 MPa y el RQD medio de los sondeos fue prácticamente del 100%.
- Materiales de derrubio, compuestos de grandes bloques de naturaleza caliza caóticamente distribuidos, inmersos en una matriz areno-arcillosa. Las propiedades de este material son prácticamente imposibles de ensayar en laboratorio y sus propiedades deben ser estimadas.
- Material brechificado que constituye la base del paleodeslizamiento. En esta capa sí se han podido realizar algunos ensayos, aunque sus resultados son poco fiables dada la dificultad de

la extracción de muestras y su consiguiente baja calidad. En él se obtuvierón límites líquidos entre 61 y 36% e índices de plasticidad entre 16 y 19. La cohesión efectiva pico medida fue de 27 kPa, mientras que la residual osciló entre 10 y 15 kPa. El ángulo de rozamiento pico fue de 15,8º, mientras que el residual osciló entre 9.6 y 18º.

Unos primeros análisis de sensibilidad indicaron que la capa que condiciona en mayor medida la estabilidad de la ladera es la de material brechificado que constituye la base del deslizamiento.

Los parámetros resistentes asocidados al nivel brechificado se han obtenido de un análisis retrospectivo, resultando una cohesión de 20 kPa y un ángulo de rozamiento de 20º.

Con estos valores se ha obtenido la superficie potencial de deslizamiento mediante un análisis bidimensional por elementos finitos con el método de degradación de c y φ. Destacar que el resultado coincide sensiblemente con el obtenido mediante métodos de equilibrio límite con círculos de deslizamiento.

2.3 *Medidas de mejora de la estabilidad*

Para la solución propuesta se ha estudiado la estabilidad de la ladera tras las medidas de mejora, obteniendo para las condiciones actuales de nivel freático, un coeficiente de seguridad de 1.4, lo que supone una mejora de la estabilidad frente a la situación actual del 40%. Los cálculos se han obtenido en un análisis bidimensional por elementos finitos con el método de degradación de c y φ.

Además de analizar la situación actual de nivel freático se han analizado otras como las de embalse lleno (coeficiente de seguridad de 1.5) o la más desfavorable de embalse vacío con presiones mantenidas en el estrato aptiense lejos de la ladera y descendiendo linealmente hasta coincidir con la cota del embalse en el cauce (coeficiente de seguridad de 1.37). No se ha calculado la hipótesis de desembalse rápido con presiones intersticiales en el relleno ya que los drenes han sido calculados para disipar éstas a una velocidad superior a la máxima prevista de descenso por vaciado del embalse.

Como se puede observar, la solución adoptada mejora la estabilidad de la situación actual, consiguiendo una garantía de seguridad suficiente para todos los casos.

3 FUNCIONAMIENTO HIDRÁULICO

3.1 *Introducción*

El estudio de soluciones llevado a cabo para las obras de estabilización de la ladera de Puebla de Arenoso puso de manifiesto desde las primeras etapas del trabajo la interacción existente entre aspectos geotécnicos e hidráulicos propios de una actuación en un cauce de río, el Mijares, que forma a su vez parte de la cola de un embalse.

En consecuencia, la singularidad de la obra ha requerido del desarrollo de una serie de trabajos encaminados a caracterizar el comportamiento hidráulico de la actuación, como son:

- Definición de los eventos hidrológicos y otros condicionantes de diseño.
- Simulación numérica mediante modelos matemáticos del comportamiento hidráulico.
- Realización de un modelo físico.

3.2 *Definición de los eventos de diseño*

A partir del estudio conjunto de los datos hidrológicos y las simulaciones de niveles en el embalse teniendo en cuenta el sistema Arenós-Sitchar, se estimó de forma aproximada la probabilidad combinada de eventos hidrológicos y niveles de embalse.

En particular, a efectos de caracterizar estadísticamente de forma simplificada las probabilidades de que el embalse se encuentre entre determinados niveles en la época de mayor probabilidad de lluvias, se ha utilizado el modelo SIMGES (Universidad Politécnica de Valencia) de manera que han sido incorporadas las principales caracterísitcas del sistema de explotación de la cuenca del Mijares utilizando una escala de tiempo mensual.

Los eventos de diseño finalmente adoptados combinan el hidrograma de 500 años de período de retorno con tres situaciones iniciales de llenado del vaso del embalse; cotas de superficie libre al inicio de la avenida de +525 m.s.n.m. (embalse vacío), +560 m.s.n.m. (pie de la obra inundado) y +580 m.s.n.m. (embalse lleno).

3.3 *Simulación numérica*

Para el caso de estudio, se elaboraron sendos modelos de simulación hidráulica unidimensionales (HEC-RAS en régimen estacionario y MIKE 11 en régimen transitorio) que permitieron obtener, para los distintos escenarios previamente definidos, un rango estimado de calados y velocidades en la actuación.

Así pues, para un evento hidrológico de período de retorno 500 años y un nivel de embalse previo por debajo de la cota 560 m.s.n.m. (con probabilidad de ocurrencia simultánea superior a 1000 años), serían esperables velocidades máximas en la actuación entre 10 y 15 m/s y calados en la misma entre 6 y 15 m.

3.4 *Modelo fisico*

De acuerdo con los resultados de las modelaciones matemáticas hidrológico-hidráulicas, las condicones de contorno morfológicas y de evolución de

niveles en el vaso del embalse, se abordó la fase de modelación física en modelo reducido, prácticamente obligatoria por:

- La tridimensionalidad y singularidad del problema, por lo que respecta a la evolución espacio-temporal de los flujos de avenida.
- La evolución del fenómeno rápidamente variado transitorio (resalto hidráulico móvil y esviado respecto al flujo).
- Las socavaciones generalizadas y/o localizadas que provoquen los remolinos turbulentos.
- Las inestabilidades de los mantos de escollera inicialmente suelta en taludes estables en seco.

En particular, se trata de un modelo físico reducido con tramos de lecho y márgenes movilizables, no distorsionado a escala geométrica 1/75, con semejanzas de Froude hidráulico y densimétrico. La longitud de estudio en la realidad (prototipo) es de unos 1200 m, lo que supone una longitud de superestructura de modelo de unos 16 m.

Como principal conclusión derivada de los distintos ensayos realizados, para el hidrograma de diseño y situación inicial de superficie libre de embalse a cota +525 m.s.n.m., se detectaron una serie de deficiencias en el funcionamiento que se sintetizan a continuación:

- Reparto de caudales desigual en la zona de aguas abajo del tajamar, de manera que se producía una concentración de flujo supercrítico hacia la margen izquierda del cauce.
- Resalto hidráulico móvil esviado respecto al flujo incidente, desplazándose hacia aguas arriba desde el final de la rampa de escollera recebada. Se producía una fosa de socavación de gran longitud que desestabilizaba la base de la escollera de protección de la ladera de margen izquierda inmediatamente aguas abajo de la obra de control, provocando un deslizamiento generalizado de la citada escollera.

Las actuaciones finalmente propuestas, agrupadas por el carácter técnico de las mismas y por su influencia sobre el comportamiento del flujo, son las que a continuación se exponen:

- Fijación de la base del talud de la margen izquierda a la salida de la estructura de control, aguas abajo, y recebado del talud de escollera de estabilización de laderas hasta la cota +574 m.s.n.m.
- Peralte transversal recebado en la pendiente de salida del 9% de la estructura de control (mejora del reparto transversal del caudal en el reintegro al cauce, para todo escenario de diseño contemplado).
- Protección areal mediante escollera recebada de la zona del lecho móvil (protección del lecho en la zona de cauce de reintegro, evitando que se produzca erosión).
- Espigones transversales de escollera, inclinados hacia aguas abajo y de longitud creciente (alejamiento de los remolinos turbulentos de la margen izquierda, centrando los caudales ordinarios del flujo).

Figura 3. Vista del modelo reducido desde aguas abajo.

4 EJECUCIÓN Y AUSCULTACIÓN

4.1 *Ejecución*

A finales del año 2005, y una vez decidida la tipología de la obra a ejecutar, comienza la primera fase de las obras (consideradas como obras de emergencia) para la ejecución de un terraplén de contención de las laderas de Puebla de Arenoso. Actualmente se está desarrollando la segunda fase de la obra, ya licitada por el procedimiento habitual.

Las unidades de obra principales consistían en los rellenos que configurarían el cuerpo del terraplén, como estabilización y aseguramiento de la ladera, y los hormigones de la estructura, para la canalización del río, evitando la erosión al pie de las laderas y "apuntalando" el terraplén dispuesto entre ellas.

Durante la primera fase se ha llevado a cabo principalmente la construcción del canal. Durante la segunda, se están concluyendo los rellenos de estabilización.

Para acometer las obras, ha sido necesario construir el canal de hormigón en dos fases, construyendo ataguías y desviando el río en dos ocasiones.

En cuanto a la descripción de las obras como tal, la solución ha consistido en la disposición de rellenos de suelos granulares, protegidos por escollera, en ambas márgenes y cauce del río Mijares, y encauzamiento de éste con una estructura en hormigón armado.

Figura 4. Desvío del río por la margen derecha. Ejecución de la mitad del canal de la margen izquierda.

Figura 5. Desvío del río por la margen izquierda. Ejecución de la mitad del canal de la margen derecha.

La actuación se ha ejecutado en el estrechamiento que presenta el embalse de Arenós frente a la localidad citada.

El relleno ejecutado sobre la ladera tiene un efecto estabilizador sobre esa margen, la del pueblo. El relleno sobre el cauce estabiliza el pie de la ladera derecha, y el de la izquierda tiene una doble finalidad: aumentar el coeficiente de seguridad al deslizamiento de esa ladera, configurándose como primera fase de posibles recrecimientos o rellenos futuros, y sirve de contrapunto frente a empujes de la ladera de enfrente.

El encauzamiento ejecutado en el tramo más estrecho como canal de hormigón armado, protege de las posibles erosiones del flujo hidráulico los pies de los rellenos de las laderas y el relleno del cauce.

El relleno sobre la ladera derecha alcanzará la cota 600, nivel máximo de explotación de la presa de Arenós, y se ha formado con un talud mínimo 2H:1V (hasta 2.35H:21V), se desarrolla en una longitud de 470 m aproximadamente, y poseerá una berma a la cota 590 en su tramo final.

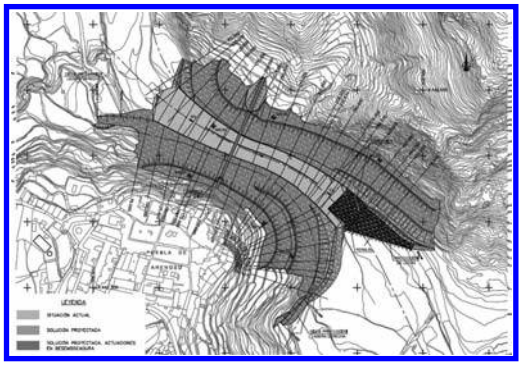

Figura 6. Planta de las obras.

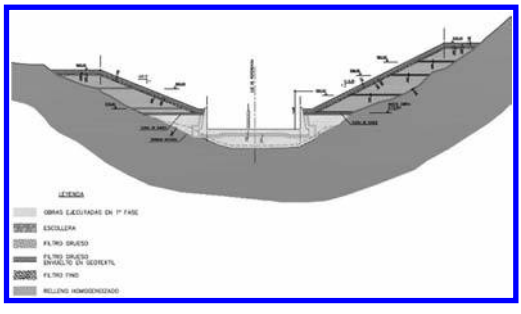

Figura 7. Sección tipo.

El relleno sobre la ladera izquierda se prevé hasta la cota 590, y se ha formado con un talud mínimo 2H:1V en su tramo final, y valor típico 2.5H:1V, sin bermas. Este relleno se desarrolla en 480 m de longitud aproximadamente.

Los rellenos de las laderas se han formado con aportación de material granular, obtenido del aluvial que el río ha depositado en margen derecha, aguas arriba y abajo del emplazamiento de la obra. Superficialmente se creó un manto de protección de escollera de 1 m de espesor, estable frente al oleaje que se puede producir en caso de avenida a embalse vacío.

Para garantizar un adecuado drenaje de los rellenos se ha dispuesto el siguiente sistema de filtros y drenes:

- Dren de ladera entre las laderas y los rellenos. En la primera fase (aproximadamente hasta la cota 574) se ha dispuesto un sistema de dren francés, consistente en unos tubos de filtro grueso horizontales envueltos en geotextil, conectados verticalmente por tubos de PVC. En la segunda fase, se está ejecutando un tapiz continuo de filtro grueso (40 cm de espesor) envuelto en geotextil. El filtro grueso hace la función de dren, mientras que el geotextil hace la función de filtro.

- Drenes horizontales, consistentes en tongadas de filtro grueso de 40 cm de espesor, envueltos en geotextil. En los rellenos de la margen derecha se disponen cuatro drenes horizontales interiores, mientras que en la margen izquierda solo dos.
- Drenes exteriores, entre los rellenos y la escollera de protección. Concretamente se disponen filtro fino y grueso.

El relleno del cauce, de hasta 5.5 m de espesor, también se realizó con el material mencionado, coronado por las capas de filtros sobre las que apoya el encauzamiento de hormigón armado.

Por otra parte, la sección de encauzamiento se desarrolla en una longitud aproximada de 190 m, y está constituido por una losa de 2.20 a 1.60 m de espesor, cajeros extremos de altura comprendida entre 8 y 10 m y espesor variable (de 1.20 m a 2.00 m). Se dispone un cajero interior de 7 m de altura y 0.8 m de espesor. La altura interior de la sección está comprendida entre 32.40 m (30.00 m libres) y 44.50 m.

El encauzamiento se ha ejecutado horizontal en los primeros 40 m y con una pendiente del 1% en los 150 m siguientes. Se disponen una rampa del 6% a la entrada, y una pendiente del 9% a la salida, que se extienden en longitudes de 55 y 120 m respectivamente.

En la primera fase de la obra se han utilizado más de 45.000 m^3 de hormigón, 2.200 toneladas de acero en armaduras y 300.000 m^3 de relleno de material granular (relleno todo uno y filtros) y escollera.

En la segunda fase se va a utilizar un total de 425.000 m^3 de material granular y escollera, que sumados a los de la primera fase suponen un total de 725.000 m^3 de volumen de tierras.

Figura 8. Vista general de la segunda fase de las obras. Obsérvese la ejecución del dren de ladera en la margen izquierda.

4.2 *Auscultación*

En la actualidad se realiza el control de los deslizamientos de Puebla de Arenoso mediante la lectura de distintos aparatos de instrumentación.

La campaña de sondeos de investigación del año 2004 permitió la instalación de 13 inclinómetros distribuidos en los dos grandes deslizamientos, incluido el pueblo.

El inclinómetro de menor profundidad es de 61 m, mientras que el de mayor profundidad alcanza los 141 m.

También se han instalado para dichas zonas un total de 16 estaciones GPS para el control de los movimientos de ladera, de las cuales dos actúan como referencia.

Las otras 14 se han repartido, 11 en la ladera de la margen izquierda y 3 en los límites del pueblo (margen derecha).

5 CONCLUSIONES

Puebla de Arenoso se sitúa sobre una lengua de material de derrubio que constituye el pie de un deslizamiento cuaternario.

Por debajo de ese material de derrubio se sitúan las margas y calizas cretácicas del Aptiense, y entre ambos materiales se ha detectado un material fluidificado de espesor decimétrico a métrico, que constituye una superficie preferente de deslizamiento.

En el estado actual, la ladera sobre la que se asienta Puebla de Arenoso se encuentra en equilibrio muy precario. Por dicho motivo se ha proyectado una obra de estabilización que consiste en un canal de hormigón de unos 30 m de anchura útil, con cajeros que varían entre 8 y 10 m, que sostienen el pie de unos rellenos que alcanzan la cota 600 (40 m por encima de la cota del cauce).

Con estas medidas se consigue un aumento del coeficiente de seguridad frente al deslizamiento del orden del 40%, respecto a la situación primitiva, es decir, antes de la actuación, obteniéndose coeficientes de seguridad para hipótesis distintas a la situación primitiva de más de 1.30.

El diseño hidráulico ha condicionado en gran manera la solución adoptada, y la realización de un modelo a escala reducida ha permitido mejorar algunas características del diseño.

Los movimientos de la ladera están siendo controlados mediante un sistema de monitorización a base de inclinómetros y estaciones GPS, que sirven de apoyo a la obra en ejecución.

Por último, cabe indicar que este control de los movimientos se mantendrá como sistema de alerta por tiempo indefinido.

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Rehabilitation of Des Plats arch dam

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ABSTRACT: The paper addresses the rehabilitation project of Des Plats arch dam. As can be observed at many dams built in middle of the last century, state of the Des Plats arch dam has been deteriorated over time and the first emptying of the dam, carried out in winter of 2005, provokes an opening of vertical joints. Due to the safety reasons, an opening of 3×3 m has been constructed at the bottom part of the dam, which makes the dam completely transparent for all water income. In order to improve the dam conditions and to satisfy the safety requirements, an important rehabilitation project has been foreseen to be carried. Analysis of several technical solutions has been carried out and the most optimized solution in terms not only of safety but also of construction schedule and technology has been proposed.

1 INTRODUCTION

Many of arch dams have been built by the middle of the last century. By 1960, 44 arch dams higher than 100 m had been commissioned and among them, 12 higher than 120 m. The scientific development in the design of the arch dams lead to an important achievement in the construction. An important step in the design of the dams was the development of the numerical methods, which enable application of 3D numerical models for static and dynamic analyses of dams. Numerical (finite element) methods open possibility to model a complete dam-foundation-reservoir system for linear and non-linear behavior. These relatively new numerical techniques become usual practice for new dams but in some cases should also be applied to the existing dams. The existing dams should be analyzed in order to check if the safety requirements are satisfied, which may also been changed since construction of the dam. Another example is rehabilitation and heightening of existing dams.

As can be observed at many dams, which had been build in the last century, some rehabilitation measures should be carried out to maintain safety at the required level and increase life time of the dam. This is especially true in case of lack or not sufficient maintenance of dams. As a first phase, an assessment of the actual state of dam should be carried out. The assessment should reply to the dam safety concerns but also to give a recommendation related to the time for optimal investment. Thereafter, an optimized rehabilitation project should be prepared, which takes into account all specificity of the project.

The paper addresses the rehabilitation project of the Des Plats arch dam. The approach can be summarized as follows: the arch dam has been analyzed taking into account the opening of the vertical joints, which occurred during the first emptying of the reservoir. Then, the rehabilitation project has been analyzed and the most optimized solution in terms of safety but also of construction schedule and technology has been recommended.

2 DES PLATS ARCH DAM

Des Plats arch dam, located close to the Saint-Etienne in the massif of Pilat (France), was built in the fifties. It is an arch dam, 21 m height (H) and the crest across the valley L = 140 m long, giving an ratio of L/H = 7 that is quite unfavorable for an arch dam. The main objective of the dam is storage of water used for supply of the city of Firminy.

After about 50 years of the dam operation, the first emptying of the dam has been carried out in winter of 2005. Combination of mechanical unloading (emptying of the reservoir) with severe thermal conditions (very cold winter) provokes an opening of the vertical joints. Following a detailed inspection of both, upstream and downstream faces of the dam, it has been observed existence of a horizontal crack crossing the dam, a complete opening of the vertical joints and traces of the calcite on the downstream face of the dam indicating a leakage and deterioration of the dam concrete. In addition to that, some of the construction (horizontal) joints were opened and an alteration of the dam faces was also observed. Detailed

analysis has been carried out by French company Freyssinet. Genesis of the cracks at the upstream face of the dam has not been clearly established. The particular geometry of the dam and results of a finite element analysis carried out for the dam design, lead to the hypothesis that the most of the cracks were created during the initial dam impounding.

Monitoring system of the dam shows its sensitivity to the temperature load. Deformation of the dam in upstream/downstream direction, induced by seasonal thermal changes, is in order of magnitude of 30 mm that is quite important in respect of the dam height.

Taking into account the general state of the dam, clearly deteriorated by the opening of the vertical joints, it has been evaluated that the dam does not meet the stability and safety requirements. Therefore, in 2006 it has been decided to put it out of service. In order to ensure evacuation of the flood $Q_{1,000}$ an opening 3 × 3 m has been created at the bottom part of the dam (Fig. 1).

Having in the mind a quite bad conditions of the dam, the Owner (Syndicat des Barrages) has decided to carry out an important rehabilitation of the dam in order to improve its conditions and to satisfy the safety requirements.

An additional hydrological analysis has been carried out and the Probable Maximum Flood Level (PMFL) has been estimated. It showed that the existing evacuation system was not enough and the dam crest can be submersed by about 50 cm. Therefore, an additional requirement was imposed related to the evacuation system, which should be revised to enable evacuation of $Q_{5,000}$, estimated to be 195 m^3/s. The existing overflow spillway, located at the central part of the arch dam, was designed to evacuate a maximum flood of 60 m^3/s.

Figure 1. Photo of Des Plats arch dam: at the right side an opening of about 3 × 3 m can be observed.

2.1 *Rehabilitation project of Des Plats dam*

It is clear that initial state of an arch dam, damaged by opening of the vertical joints, will be very difficult to re-establish. A constructive connection between the block should be re-established by additional shear keys and additional joint grouting. Construction of additional shear keys at an existing arch dam is challenging task but can be solved. A similar task has been carried out on Les Toules arch dam in Switzerland (Cekerevac & Wohnlich, 2010). However, the complete state of the dam and doubt about possibility to re-establish the arch effect in the existing dam lead to the requirement to study more radical rehabilitation measures.

Therefore, an analysis of several technical solutions has been carried out with the aim to choose the best solution, from technical and economical point of view. The analysis showed a clear advantage to transform the existing arch dam to the gravity dam by adding a downstream part made of a RCC section (Figs. 2–3). The dam will then become a gravity dam with almost vertical upstream face and downstream slope of 0.7/1.0 (horizontal/vertical). The two sections of the dam, existing arch dam and new RCC section, will be connected by reinforcement bars designed to ensure the dam behavior as monolithic gravity dam. Such solution enables to erect the overflow spillway at the central part of the dam and to ensure the dam water-tightness by the existing arch dam (Figs. 4–5). The overflow part is composed of four passes, each 12 m wide, which ensure the evacuation of the PMFL (Fig. 3). The crest elevation at the overflow part (917 masl) corresponds to the normal operation water level

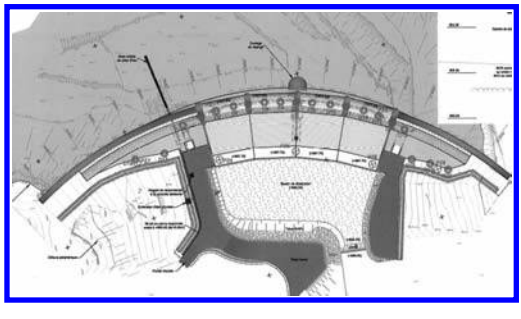

Figure 2. Rehabilitation of Des Plats arch dam, plan view.

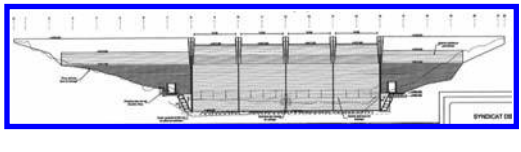

Figure 3. Rehabilitation of Des Plats arch dam, downstream view.

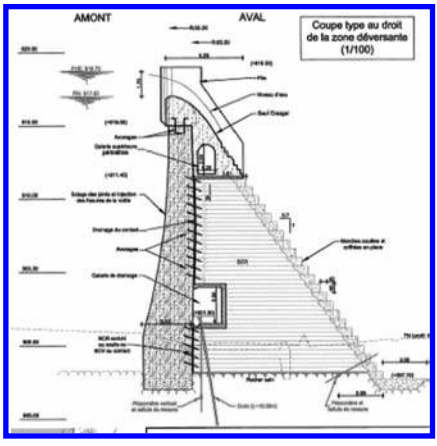

Figure 4. Rehabilitation of Des Plats arch dam, cross section at the central, spillway section.

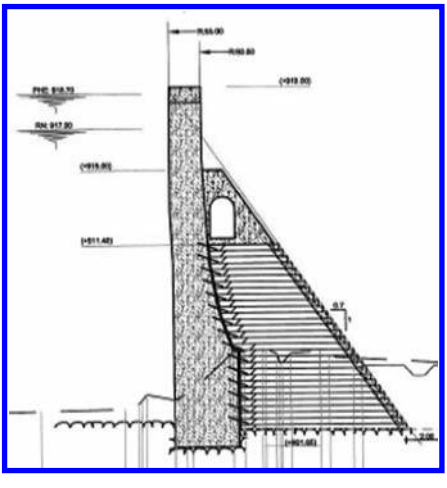

Figure 5. Rehabilitation of Des Plats arch dam, typical cross section at the lateral parts.

while the dam elevation is proposed to be heightened by 2 m (919 masl). The normal water level remains the same as for the existing dam, while the maximal flood water level ($Q_{10,000}$) has been fixed at el. 918.7 masl in order to avoid an influence of the reservoir on the existing upstream road and housing.

In addition to the above presented rehabilitation measures, it has been foreseen to saw the vertical joints of the arch dam. In order to ensure watertightness of the dam, a geomembrane should be erected at the upstream part and the existing cracks should be injected. Foundation of a new, rehabilitated dam will be much larger and therefore a new drainage system is foreseen to be built from downstream, drainage gallery (Fig. 4). The drainage gallery will also be used to collect any leakage through the upstream part of the dam. The existing water intake and the plunge pool should also be refurbished.

The above presented rehabilitation project is under approval procedure and can slightly be modified based on remarks and specific requirement made by the Owner.

3 STATIC AND DYNAMIC ANALYSIS OF THE REINFORCED DES PLATS DAM

This chapter presents the main assumptions, methodology and results of the stability, static and thermal stress analyses of the reinforced arch dam. The reinforcement of the existing dam is accomplished by constructing an RCC abutment at the downstream side of the dam. The RCC reinforcement will be founded on healthy rock and will thus convert the existing arch dam into a gravity dam. The connection between the existing dam and the proposed reinforcement will be attained by means of passive anchorages whereas on the upper part of the RCC a pier of conventional mass concrete is considered which will connect the two structures.

The aforementioned analyses have been based on currently available data and complementary assumptions usually used in the dam design practice. The calculations were made with finite elements method by using a 2D cross section of the reinforced dam, composed of three different concrete types (Fig. 4). The dam foundation is at elevation +897.00 masl and the dam crest at +919.00 masl. The dam has almost vertical upstream facing and a 1V:0.7H downstream facing inclination.

3.1 *Material properties*

In the present analysis, linear elastic and isotropic homogeneous material properties of the concrete were adopted (Table 1). The assumed properties are mainly based on existing in-situ and laboratory data. However, due to the lack of some specific tests, a few properties were assumed based on our experience gained on similar projects.

Table 1. Mass concrete properties used for the static and thermal analyses.

	E [GPa]	Thermal dilatancy [1/°C]	Heat conductivity [kN/day/°C]
New concrete	25	0.8e-5	240
Existing concrete	20	1.25e-5	240
RCC	25	0.75e-5	225
Dam foundation	15	0	240

The rock foundation of the dam is composed of granites with generally good mechanical characteristics. Therefore, we have considered a modulus of elasticity of the rock mass of 15 GPa and a friction angle of $\phi = 45°$. Detailed geological survey of the site showed no any major faults and presence of some joints which can be classified into two main groups (Fig. 6): a sub-vertical joint set (dip from 78° to 90° and variable dip direction) and a sub-horizontal joint set (dip around 10° and variable dip direction).

3.2 *Load combinations and safety criteria*

The load cases considered for the static and stability analyses of the dam are based on the recommendations given by ICOLD. The load combinations considered in the present study were limited to: thermal, usual static and extreme static for summer and winter temperature variations. The load cases used for each load combination are shown in the Table 2.

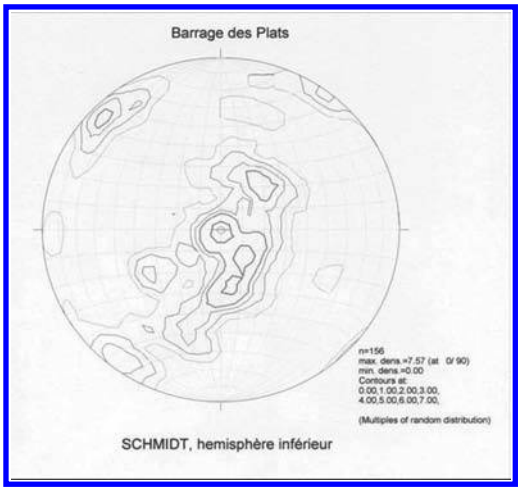

Figure 6. Des Plats dam—Schmidt presentation of the joint sets—lower hemisphere.

Table 2. Design load cases and load combinations.

Load case	Self weight	Temperature		Water pressure		
		Summer	Winter	Empty	NWL	PMFL
Thermal	x	x		x		
	x		x	x		
Static usual	x	x			x	
	x		x		x	
Static extreme	x	x				x
	x		x			x

3.3 *Stability analysis*

The dam stability is analyzed in respect to: safety against sliding along the contact concrete-rock foundation, safety against overturning and safety against uplifting. The analyses have been carried out according to the recommendation given by French Committee of Dams (Comite Technique Permanent des Barrages). A partial safety factors for cohesion and friction angle has been used, which depends also on the load case (Table 3).

The stability analysis of the dam is performed on a 2D model of the dam-foundation-reservoir system. The performed calculation was used to evaluate the global stability of the dam as well as the stresses at the upstream and downstream edges of the contact concrete-rock foundation. The dynamic analysis is performed by means of the pseudo-static method considering a Peak ground acceleration of $a_x = 0.11$ g. The seismic analysis has been performed in two phases:

– The stress analysis, carried out by using the peak acceleration (spectral acceleration for pseudo-dynamic method);
– Stability analysis using sustained (spectral) acceleration values taken as 2/3 of the peak accelerations values. Due to the nature of the dynamic loads (cyclic loading with relatively high frequency and change of the load direction) the stability analysis has been carried out by using this, lowered values of the ground acceleration.

Results of the safety analyses (against sliding, overturning and uplift) reveal that the reinforced Des Plats dam satisfy the required safety factors for all load combinations. In addition to that, stresses at the contact dam-foundation have been calculated by means of the beam theory. Compressive stresses, calculated at the downstream edge of the contact, ranges between 0.3 and 0.4 MPa. The maximal tensile stress of 0.05 MPa has been calculated at the upstream limit of the dam-foundation

Table 3. Required safety factors for analyzed load combinations.

Load combination	Fs against sliding	Fs foundation-concrete	Concrete stress safety factors	
			Compressive	Tensile
Usual	Fc = 3.0 Ftgϕ = 1.5	Fc = 3.0	3.0	3.0
Static extreme	Fc = 2.0 Ftgϕ = 1.2	Fc = 2.0	3.0	3.0
Dynamic extreme	Fc = 1.0 Ftgϕ = 1.0	Fc = 1.0	1.0	1.0

contact. Therefore, the calculated (compressive and tensile) stresses remain below the yield limits. More detailed analysis of the stresses in the dam body is performed by means of finite element calculations, presented in the next chapter.

3.4 *Static and dynamic stress and strain analysis*

The analysis is carried out by means of the finite-element method. The computations are performed by means of the software Z_Soil (www.zace.com).

Both the existing and the new part of the dam sections have been analyzed by using the finite element method and assuming plane strain conditions. Linear elastic behavior is assumed for both dam and foundation materials. To avoid side effects, domain borders were defined at about 5 times the dam height. In the FE model, presented in Figure 7, the dam-foundation interface was modeled through contact elements having a friction angle of $\phi = 45^{o}$.

The following load combinations were considered:

a. Usual static: Self weight + Thermal stresses + hydrostatic pressure at NWL (917 masl);
b. Extreme static: Self weight + Thermal stresses + hydrostatic pressure at PMFL (918.7 masl).

For the calculation of the thermal stresses, four different temperature variations on the dam faces were considered, depending on the different seasonal temperatures and the water level in the reservoir:

1. Variation from the mean annual to the mean maximum summer temperature (+25°C) considering an empty reservoir;
2. Variation from the mean annual to the mean minimum winter temperature (–10°C) considering an empty reservoir;

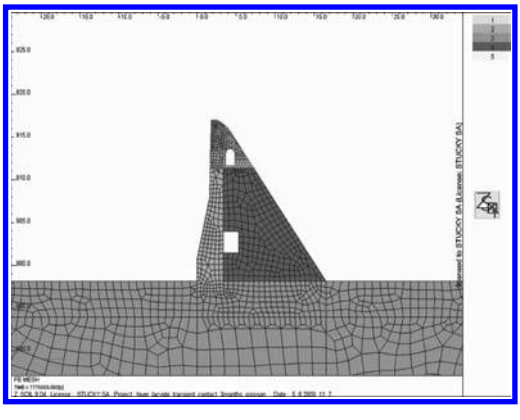

Figure 7. Global view of the dam-foundation finite element model.

3. Variation from the mean annual to the mean maximum summer temperature (+25°C) considering the NWL water level (917 masl);
4. Variation from the mean annual to the mean minimum winter temperature (–10°C) considering the NWL water level (917 masl).

For the above mentioned temperature variations the extreme temperature gradients are as follows:

Tmax = T(summer) – T(reference) = 15°C
Tmin = T(winter) – T(reference) = –15°C

Regarding the seasonal variation of the water temperature in the reservoir, a constant temperature at the depth of the reservoir was considered, being equal to 5°C in summer and 4°C in winter. Whereas at the water surface the extreme temperature values were estimated based on regional temperature data as follows:

Tmin, water = 0°C for the winter and
Tmax, summer = 15°C for the summer.

3.5 *Summer empty reservoir*

The temperature distribution in the dam was calculated by means of transitory thermal analyses for a time interval of one month considering the heat properties presented in Table 1. The ground temperature from a certain depth and on was considered to be constant and equal to the mean yearly temperature (10°C).

Figure 8 presents the temperature distribution at the end of the first month as well as the temperature evolution in the core and the faces of the dam. Figure 9 presents distribution of vertical stresses and distribution of the principal stresses due to temperature variation.

Analysis of calculated stresses reveals that quite high tensile stresses appear both in the new concrete (2.7 MPa) and in the RCC (1.5 MPa). The tension is solely concentrated in the vicinity of the galleries as presented in Figure 9 and it greatly attenuates as we move towards the core of the dam. The calculated compressive stresses are quite low and the maximal value of 3 MPa has been calculated in the existing (arch dam) concrete.

3.6 *Winter empty reservoir*

For this load case, a thermal gradient of –15°C was imposed over a period of 1 month prior to the mechanical loading (self weight). Distribution of temperature and vertical stresses at the end of the thermal loading phase is presented in the Figure 10. Contrary to what we have observed in the former case (summer empty reservoir), the maximal tensile stresses for this load case are located near the two faces of the dam. The maximal tensile stress

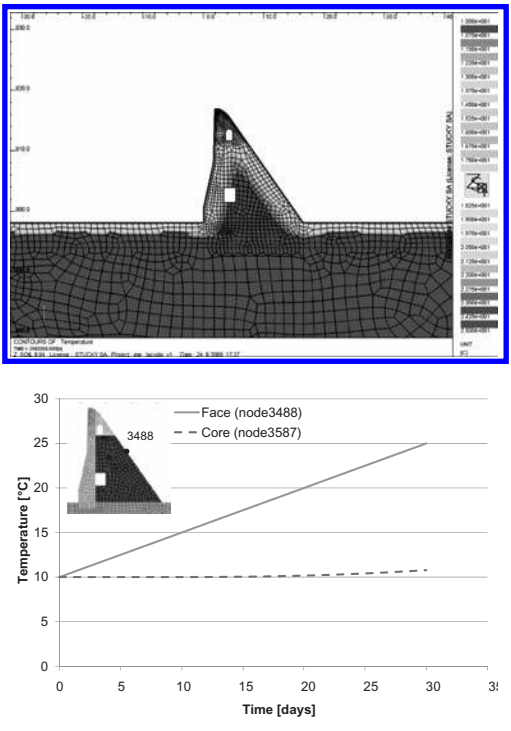

Figure 8. Temperature distribution at the end of the first month (upper Figure); temperature evolution in the dam for the load case "summer empty reservoir" (lower Figure).

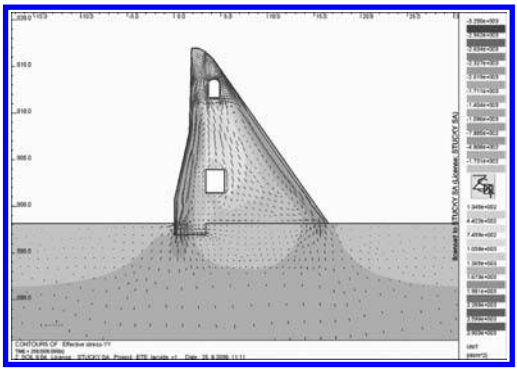

Figure 9. Vertical stresses and direction of the principal stresses due to temperature variation.

has been calculated at the upstream face (2.3 MPa) and slightly lower value the downstream RCC section (2.0 MPa). These relativelly high values of the tensile stresses can be observed only in the close vicinity of the upstream and downstream face. An important decrease of the stresses can be observed in the core of the dam.

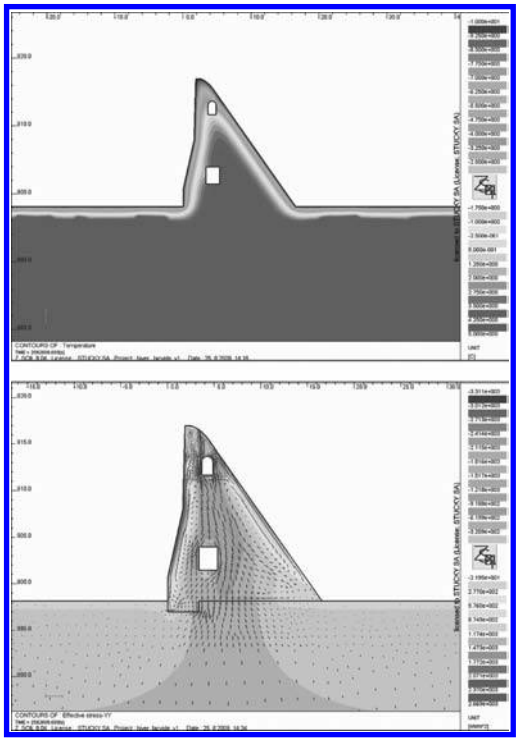

Figure 10. Temperature distribution at the end of the first month (upper Figure); Vertical stresses and direction of the principal stresses due to temperature variation (lower Figure).

3.7 *Summer temperature full reservoir*

In addition to the thermal load (summer) a hydrostatic pressure, which corresponds to the full reservoir level (918.7 masl) had also been taken into account. The analysis has been carried out in two steps. In the first step, thermal analysis has been carried out (Fig. 11). Corresponding stresses and deformation were calculated with assumption of a linear elastic relationship between strain and the temperature variation. Thereafter, the mechanical loading has been applied to already deformed mesh (Fig. 11). The maximal tensile stresses of about 1.2 MPa are located in the vicinity of the gallery. The calculated stresses are much lower in comparison with the "summer empty reservoir" load case.

3.8 *Winter full reservoir*

Similarly to the former case, thermal gradient that corresponds to the winter conditions has been imposed prior to the mechanical loading. Temperature distributions over time and corresponding thermal stresses and strains have been calculated. Thereafter, mechanical loading has

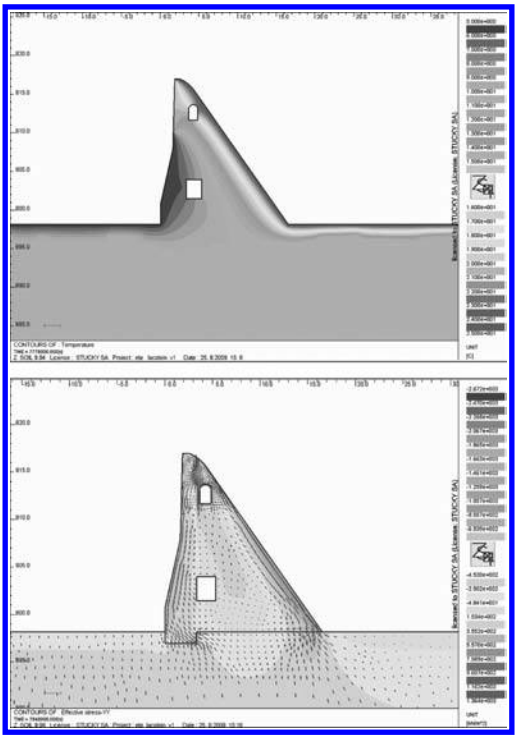

Figure 11. Temperature distribution at the end of the thermal load phase (one month - upper Figure); Vertical stresses and direction of the principal stresses due to "summer temperature full reservoir" load case (lower Figure).

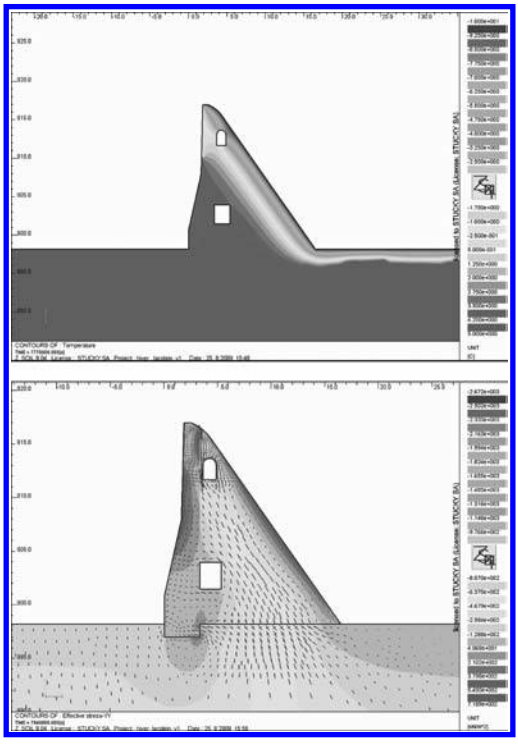

Figure 12. Temperature distribution at the end of the thermal load phase (one month - upper Figure); Vertical stresses and direction of the principal stresses due to "winter temperature full reservoir" load case (lower Figure).

been introduced, which produce additional strains and stresses in the dam body. The maximal tensile stresses in this case are quite low (0.7 MPa) and are located near the two faces of the dam (Fig. 12).

The required strength of the mass concrete has been calculated based on the bi-axial principal stresses produced in the dam. The maximum tensile and compressive stresses were calculated for the different concrete zones and for the above mentioned load conditions and load cases. Based on that, the concrete required strength was calculated for all the aforementioned load cases and load conditions taking into account the stress safety factors given in Table 3. Finally, the critical (highest) values have been assumed. Required strength for new concrete is 28.5 MPa and for RCC section 20.5 MPa.

3.9 *Interface between existing and new concrete*

In order to design interface between existing, arch dam, and new RCC section the monolithic response of the reinforced arched dam was analyzed for all the above mentioned load cases. More specifically, for each load case the imposed shear resultant on the interface between the two part of the dam was calculated and then following the methodology described in the building code ACI 318-95 (1999) the number of required passive anchorages was determined for the most unfavorable load case. The design of the interface was implemented in a way that all parts of the dam will behave monolithically.

The highest shear stresses were calculated for the load case "winter empty reservoir" (Fig. 13). In order to ensure monolithic contact between the different parts of concrete, passive anchorages of ϕ32 perpendicular to the interface were chosen. By applying provisions given in the directives of the ACI building code a number of 33 passive anchorages is required. These result to one anchorage every 55 cm (in every second layer approximately) in the vertical direction and every 1 m in the horizontal direction.

An alternative approach is the use of inclined anchors to compensate for parts of the shear force by the vertical components of the anchor force. Use of anchors of 45° angles allows reducing the number of anchors to 23 with a spacing of

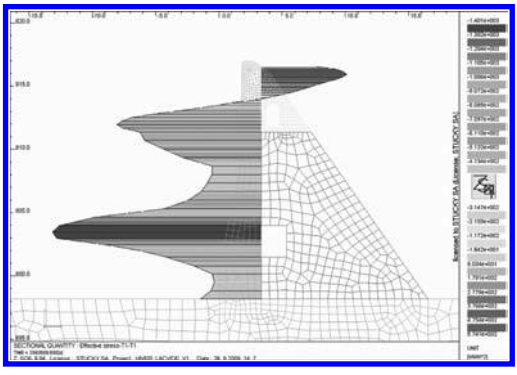

Figure 13. Shear stresses of the interface arch dam—RCC section for the load case "winter empty reservoir".

about 80 cm in vertical and 100 cm in horizontal direction.

4 CONCLUSION

A methodology employed for the preparation of the rehabilitation project of Des Plats arch dam has been presented. As can be observed at many dams built in middle of the last century, state of the Des Plats arch dam has been deteriorated and the first emptying of the dam carried out in winter of 2005, provokes an opening of the vertical joints. Following a detailed inspection of both, upstream and downstream faces of the dam, it has been observed existence of horizontal cracks crossing the dam, a complete opening of the vertical joints and traces of the calcite on the downstream face of the dam indicating a leakage and deterioration of the dam concrete. In addition to that, some of the construction (horizontal) joints were opened and an alteration of the dam faces was also observed. Due to the safety reasons, an opening 3×3 m has been created at the bottom part of the dam which makes the dam completely transparent for all water income. An important rehabilitation project has been foreseen to be carried out in order to improve its conditions and to satisfy the safety requirements. Additional hydrological study has shown that the Probable Maximum Flood Level (PMFL) is higher than actually assumed and therefore, the rehabilitation project has to integrate a new overflow system. Analysis of several technical solutions has been carried out and the most optimized solution in terms not only of safety but also of construction schedule and technology has been proposed.

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Obras para corrección de subpresión y adecuación del cimiento y sistema de drenaje de la presa de Cancho del Fresno en Cañamero, Cáceres

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RESUMEN: La vigilancia de la presa de Cancho del Fresno mostró un aumento de la subpresión en su cimiento. Se proyectó subsanar este problema y de manera complementaria, llevar a cabo una importante actuación en la presa. De esta manera las obras acometidas se componen de las siguientes fases: Un tratamiento previo de consolidación, una nueva pantalla de impermeabilización y la regeneración del plano drenante de subpresión. Se complementan las obras con la instrumentación de los nuevos drenes y la sustitución de los piezómetros de cuerda vibrante.

ABSTRACT: The Cancho del Fresno dam monitoring showed an increase in uplift pressure beneath its foundation. Actions were carried out to solve this problem and in a supplementary way, to make an important upkeep at the dam. Works included the following phases: A pre-treatment for foundation consolidation, a new grouting curtain to reduce seepage and drainage regeneration. Works are complemented by new instrumentation, new drains and the replacement of the vibrating string piezometers.

1 INTRODUCCIÓN

El embalse de Cancho del Fresno está situado en el término municipal de Cañamero en la provincia de Cáceres. La presa se ubica sobre el río Ruecas, en el término municipal de Cañamero (Cáceres).

La principal finalidad de la presa de Cancho del Fresno es abastecer de agua a las poblaciones de Cañamero, Logrosán y Zorita, laminar las avenidas del río y utilizar sus excedentes para la utilización en los regadíos existentes aguas abajo. Con la posterior construcción de las presas de Ruecas, Sierra Brava, Gargáligas y Cubilar el embalse de Cancho del Fresno pasó a formar parte del sistema de explotación hidráulica de la zona regable del Canal de Las Dehesas.

Se trata de una presa de gravedad de hormigón vibrado, planta recta y perfil triangular. El paramento de aguas arriba tiene talud 0,05 y el de aguas abajo 0,75. La altura sobre cimientos de la presa es de 57 m. La coronación está a la cota 629,50 m y su longitud es de 217,00 m. El ancho es de 9 m. El cuerpo de presa se divide en 15 bloques mediante 14 juntas verticales encofradas. La impermeabilidad de las juntas se logra mediante dos bandas paralelas de PVC situadas en el paramento de aguas arriba. Los bloques tienen un ancho de 15 m salvo el B7 y el B8 (12 m) y no se disponen juntas longitudinales.

Figura 1. Presa de Cancho del Fresno.

2 SITUACIÓN PREVIA DE LA PRESA

Vamos centrarnos en los datos de las filtraciones observadas y a los valores de la subpresión en el cimiento. Para ello nos basaremos en la "Nota Informativa de la Situación de las Presiones Intersticiales en el Cimiento de la Presa de Cancho del Fresno" de Marzo de 2006, realizado por la empresa OFITECO, y en las observaciones realizadas por el equipo técnico de explotación de la presa.

La Nota Informativa mencionada pone de manifiesto las siguientes conclusiones:

a. La situación de las presiones intersticiales en el cimiento, que en este caso pueden ser consideradas como subpresiones por la cercanía de los puntos de control al contacto presa-cimiento, no puede considerarse satisfactoria puesto que son elevadas. Del análisis llevado a cabo a partir del estudio de los gráficos de evolución temporal y de la modelización estadística se puede decir que se aprecia una importante influencia de las variaciones del nivel del embalse en los puntos de control situados más aguas arriba, siendo más moderada en los de más aguas abajo, como era de esperar.
b. En la zona inmediatamente aguas abajo de la pantalla de drenaje se han alcanzado porcentajes de la presión con respecto a la carga hidrostática en el año 2005 del orden del 75% en el bloque 8 y del 78% en el bloque 9, que son bastante elevados, lo que indica que los drenes no están aliviando subpresiones como sería deseable. Además, la tendencia observada en los últimos años indica que esta relación seguirá aumentando, pues parece que la red de drenaje está inmersa en un proceso de colmatación (por carbonatos principalmente), taponándose las grietas y perforaciones realizadas, que hará que los drenes acaben siendo totalmente ineficaces, comprometiendo la estabilidad de la estructura debido al incremento de la subpresión.
c. En los bloques 8 y 9, junto al paramento de aguas abajo, se están registrando presiones con respecto a la carga hidrostática que superan incluso la ley teórica triangular correspondiente a la situación de drenes ineficaces (como si no existiera la pantalla de drenaje), con valores del orden del 45% en el primer caso y cercanos al 40% en el segundo, que son muy elevados para la ubicación en la que se encuentran los puntos de control. También en estos puntos se está produciendo un continuo y lento aumento a lo largo de los últimos años, muy similar al que se ha registrado en los puntos de control situados aguas arriba.
d. De la modelización estadística realizada con los valores de los piezómetros del bloque 9 se puede decir que en el periodo de comprobación de los modelos, entre Julio de 2004 a finales de 2005, los valores medidos en muchos casos superan a los esperados. En el punto situado aguas abajo de la pantalla de drenaje (PCV-3), se ha obtenido una deriva de +0,21 m.c.a./año entre los años 2001 y 2005.

Entre las recomendaciones señaladas en la Nota Informativa, además de continuar con la realización de lecturas o la posibilidad de instalar nuevos piezómetros, están:

1. La limpieza de los drenes de la galería inferior utilizando una hidrolimpiadora, y en caso de que este proceso no fuera efectivo, se debería llevar a cabo una reperforación de estos drenes mediante una sonda. Posteriormente, se debe analizar si la subpresión en los dos bloques controlados ha disminuido lo suficiente; en caso contrario, se recomienda la realización de nuevas perforaciones.
2. Estudiar la posibilidad de realizar una campaña de impermeabilización del cimiento que refuerce la existente, a partir de inyecciones desde la galería inferior hacia la zona de aguas arriba de la pantalla de drenaje.

La Nota Informativa concluye poniendo de manifiesto el gran volumen de filtraciones y el alto nivel de subpresiones, y la necesidad de redactar un Proyecto de Tratamiento de la Cimentación en la Presa de Cancho del Fresno.

Del examen directo de la presa se concluye que una parte muy importante de la filtración tiene lugar a través de los drenes, donde se puede observar la emigración de la arcilla hacia las canaletas de la galería perimetral de la presa.

En cuanto a las filtraciones en las galerías provenientes de las juntas verticales, no se observan caudales importantes, debiéndose en su mayor parte al aporte realizado a través de los drenes.

3 REDACCIÓN DE PROYECTO

Con estos datos la Confederación Hidrográfica del Guadiana toma la decisión de redactar un proyecto para subsanar los problemas y adecuar la presa a una correcta y segura explotación de la misma.

El objeto de la ejecución del proyecto es la realización de los trabajos de inyección necesarios para reforzar tanto la *consolidación* del contacto presa—cimiento como la *pantalla de impermeabilización* en el terreno mediante la inyección de lechadas de cemento a través de taladros en profundidad. Además se proyecta una *nueva red de drenaje* de subpresión después del refuerzo de la pantalla, con la renovación del sistema de *auscultación* y *se mejora*

el desagüe que evacua las aguas recogidas en el nivel 1 y va hasta el paramento aguas abajo, con una reperforación del conducto existente.

4 DESCRIPCIÓN DE LAS OBRAS

Las condiciones de trabajo son, por decirlo de una manera elegante, muy particulares. Se trabaja en el interior de galerías con unas dimensiones de 1,80 m. de ancho por 2,50 m. de alto en la clave de la misma. Los equipos de trabajo son pesados y muy ruidosos y se tienen que desplazar por galerías y escaleras estrechas de forma manual.

Hay que hacer una importante instalación previa de la obra. Tanto la fabricación de lechadas como la acumulación de detritus de perforación se realizan en el exterior. Así como la generación de electricidad, el suministro de aire para los martillos neumáticos, el suministro de agua y líneas de comunicación para los operarios. Para ello es preciso tender una serie de líneas de conducción, cables, mangueras y latiguillos de diferentes diámetros a lo largo de las galerías de la presa, tanto en sentido entrada como en sentido salida de la presa.

Es muy importante en este tipo de obras la organización previa de los trabajos y tener los tajos siempre limpios y ordenados para evitar accidentes y tener una buena productividad.

4.1 *Tratamiento de consolidación*

El refuerzo de la consolidación de la presa se llevó a cabo mediante la perforación e inyección de taladros de Ø 46 mm. realizados desde la galería perimetral y los pasillos del nivel 1, cubriendo todo el apoyo en planta de la presa. S realizó mediante la perforación y posterior inyección de 189 taladros tanto verticales como inclinados para alcanzar la superficie de apoyo de la presa. En la galería perimetral se han realizado perforaciones verticales cada 1,5 metros con una profundidad media de 11,50 metros. En los pasillos del nivel 1 se han realizado perforaciones tanto verticales como inclinadas. Se han completado 31 taladros verticales con una profundidad media de 17,05 metros y 40 perforaciones inclinadas con una profundidad media de 20,40 metros. En la realización de las inyecciones no se atravesaron las juntas transversales existentes entre bloques.

La longitud de los taladros para las inyecciones de consolidación sumó un total de 2.755,10 m.

La cantidad final de cemento inyectado fue de 275,50 Tn.

4.2 *Refuerzo de la pantalla de impermeabilización*

El refuerzo de la pantalla de impermeabilización se llevó a cabo mediante la perforación e inyección

Figura 2. Perforación para tratamiento de consolidación.

de taladros de Ø 46 mm. realizados desde la galería perimetral y con una distancia entre taladros de 3 m en los bloques 3 a 6 y 12 a 14, y de 1,5 m entre taladros en los bloques 7 a 11. La profundidad de los taladros está comprendida entre 17,50 y 45,70 m, dependiendo si se han realizado en los bloques centrales de la presa, los más profundos, o en los bloques laterales. Pasado el contacto de la presa con el terreno y alcanzada la profundidad definida para cada punto se comenzaba una inyección en retroceso obturando en tramos de no más de 4 metros.

En total se han realizado 79 perforaciones para el tratamiento de impermeabilización sumando una medición de 2.622,81 metros.

La admisión final de cemento fue de 365,95 Tn.

Los trabajos de inyección son sumamente delicados. En ningún caso se han de sobrepasar las presiones y caudales de inyección definidos en proyecto. La consecuencia puede llegar a ser justamente lo que se pretende evitar, generar subpresiones tales que comprometan la estabilidad o integridad de los bloques inferiores.

En el proyecto que nos ocupa el protocolo para la fase de consolidación comprendía caudales no superiores a 15,00 l/min. y presiones comprendidas entre 2,00 Kg/cm^2 y 4 Kg/cm^2, teniendo marcada como presión de cierre máxima 5,00 kg/cm^2.

Para la pantalla de impermeabilización los valores de caudal eran los mismos, si bien se aumentaba la presión de inyección para las obturaciones

Figura 3. Equipo de inyección.

Figura 4. Caudalímetro y captador de presión digitales.

profundas, por debajo del contacto hormigón-roca. Para las obturaciones de mayor profundidad, rondando los 40 metros, se aumentaba la presión hasta una presión máxima de cierre de 12,00 kg/cm^2.

Para controlar y tener el rigor necesario en las operaciones de inyección se utilizaba un equipo autónomo con medida instantáneo de la presión y el caudal de inyección en boca de taladro.

4.3 *Nueva pantalla de drenaje*

Dado que el estado de la red de drenaje existente se encontraba colmatado en gran parte, conviene proceder a la inyección de los drenes y a la perforación de una nueva pantalla de drenaje realizada aguas abajo de la de impermeabilización. Los drenes se realizaron con diámetro 80 mm. y una profundidad variable tal que quede en torno a 3,00–4,00 m por encima de la pantalla de impermeabilización, salvo en la parte de cauce de la presa que la diferencia de profundidad podrá llegar a ser de 24 metros. Se perforaron 5 drenes por bloque de 15 m y 4 en los de 12 m, de forma que queden separados cada 3 m. Los drenes.

Figura 5. Galería y drenes a la finalización de los trabajos.

Han quedado rematados con un tubo de acero galvanizado de 1 metro de longitud alojado en el hormigón de la presa acabado en una pieza roscada al tubo y con una boca de enchufe rápido. Todo ello dentro de una pequeña roza cubierta por un tramex para evitar que se ensucie y favorezca la salida de agua al las canaletas de la presa. En la boca de enchufe rápido se puede conectar un tubo de acero inoxidable dotado con grifo y manómetro para poder aforar y medir la presión de los drenes.

En total se han perforado 56 drenes con una longitud total de 1.225 m.

4.4 *Reposición de la auscultación*

En la unidad para la reposición y mejora de la auscultación, se han incluido la instalación de 8 Piezómetros de Cuerda Vibrante, 4 sustituyendo unidades antiguas y otros 4 en nuevos puntos de control, 4 aforadores tipo Thomson y 16 medidores tridimensionales de juntas.

4.5 *Mejora del desagüe para evacuar las aguas del nivel 1 y 2*

Se ha llevado a cabo una reperforación y limpieza de los conductos existentes en la presa para el desagüe de las aguas procedentes de los drenes de subpresión y de cuerpo de presa. En total han sido dos los conductos tratados. Por un lado el que da salida a toda el agua proveniente de los drenes, alojado en el nivel 1 de la presa. El otro conducto tratado es el de desagua la cámara de válvulas.

5 RESULTADOS Y CONCLUSIONES

Una vez concluida la obra se retoma la medida automática de los piezómetros de cuerda vibrante. Esta ha volcado unos resultados muy satisfactorios.

Recordando los valores de las cargas que reflejaban los piezómetros en los bloques 8 y 9 junto a la pantalla de impermeabilización, 75% y 78% respectivamente, encontramos en las últimas lecturas valores de 22% y 25% respectivamente.

De igual manera, si comparamos los valores que teníamos en los puntos de control próximos al paramento de aguas abajo, nos encontrábamos para los bloques 8 y 9 valores de un 45% y 40%, teniendo en la actualidad lecturas que rondan respectivamente un 31% y 33% de carga.

Es importante reseñar que los valores actuales están tomados con el embalse al 100% de su capacidad, mientras que los del 2005 al 55%, lo que da más importancia aún a la mejora conseguida.

Menos riguroso, pero de mayor impacto visual, es la impresión tras una visita a las galerías. Una vez concluidos los trabajos habían desaparecido las filtraciones de agua por los estribos de roca vista en los distintos niveles, de la misma manera cesaron las filtraciones por las juntas longitudinales, en la cota del suelo, de las galerías transversales de la presa del nivel 1.

La obra se ejecutó en estrecha colaboración y siempre apoyados por el buen criterio técnico del Director de Obra y su equipo de ingenieros, todos miembros de la Confederación Hidrográfica del Guadiana.

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Rehabilitación integral de la presa y embalse de Proserpina

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RESUMEN: La presa de Proserpina es una construcción de indudable origen romano, vinculada al abastecimiento de agua a la antigua *Emerita Augusta*. Perdida esta función de abastecimiento, la presa ha seguido sin embargo en servicio utilizándose su agua para diversos usos. Ello ha implicado un mantenimiento continuado de la presa, con múltiples actuaciones que solo comienzan a estar documentadas a partir del siglo XVI. A mediados del siglo XX la presa es puesta en uso por los Servicios Hidráulicos del Estado, y en la década de 1990 a 2000 se realizan sobre la presa actuaciones de gran importancia. Recientemente se están finalizando por parte de la Confederación Hidrográfica del Guadiana y el Ministerio de Medio Ambiente unas actuaciones que han supuesto una rehabilitación integral de la presa y su entorno, mejorando sus condiciones de seguridad y funcionalidad y facilitando su visita turística, actuaciones que se describen en esta comunicación.

ABSTRACT: Proserpina Dam is a construction of clear Roman origin, related to water supply to the ancient *Emerita Augusta*. When this use was lost, the dam continued in service, its water being used for various purposes. This has involved the ongoing maintenance of the dam, with multiple performances which are only documented from the sixteenth century on. In the mid of twentieth century, the dam is put into use by the State Hydraulic Services, and in the decade from 1990 to 2000 are made important works on the dam. Recently the State Hydraulic Services (Guadiana river basin Authority and Ministry of Environment) are conducting a comprehensive rehabilitation of the dam and its environment, improving their safety and facilitating functionality and sightseeing, actions which are described in this paper.

1 INTRODUCCIÓN

En las dos últimas décadas se han adquirido conocimientos muy importantes sobre la presa de Proserpina, habiéndose escrito y hablado mucho sobre la misma, a veces sin demasiado acierto, por lo que es difícil resumir en unas líneas lo que es esta presa y dar algunas nociones sobre sus casi 2000 años de historia (para ampliar información sobre el tema ver Martín Morales, Aranda Gutiérrez y Sánchez Carcaboso, 2002).

Sin duda alguna, y como veremos recientes hallazgos así lo demuestran, la presa era el origen de uno de los tres sistemas de abastecimiento de agua con los que llegó a contar la Mérida romana. (Aranda Gutiérrez, Sánchez Carcaboso, Andrés Díaz *et al*, 2008).

La presa tiene una longitud de unos 430 m y una altura máxima de unos 21 m, estando compuesta por dos elementos estructurales de diferente naturaleza pero que trabajan conjuntamente: por una parte un muro-pantalla constituido por diversas fábricas pétreas (sillería, mampostería y hormigón romano ("*opus caementitium*", denominado localmente "calicanto") y reforzado por contrafuertes tanto aguas arriba como aguas abajo, y por otra un terraplén o espaldón de tierras adosado al muro aguas abajo del mismo.

Cuenta con dos torres de toma, denominadas localmente "bocines", correspondiendo el situado hacia la margen derecha a una toma profunda o desagüe de fondo, y el situado hacia margen izquierda a una toma más elevada, de la cual por comparación de cotas con los restos existentes del acueducto, se suponía (y ahora se ha comprobado) que debía partir éste. Ambos bocines son estructuras de piedra, de planta casi cuadrada, adosados por un lado al muro de la presa y por los otros tres al espaldón, por lo que su aspecto es realmente el de un de pozo, contando con unas escaleras para descender al fondo, teniendo el de margen derecha una profundidad de unos 18 m, y el de margen derecha de unos 10 m.

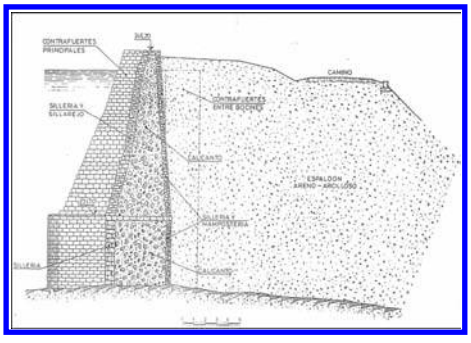

Figura 1. Sección tipo de la presa de Proserpina.

Estos bocines están conectados por un lado con el embalse mediante conductos (originalmente tuberías de plomo), con los elementos de apertura-cierre de los mismos alojados en el propio bocín, y por tanto accesibles para su manejo. Por la cara opuesta, de los mismos parten unas galerías que discurren bajo el espaldón permitiendo salir fuera de la presa a los caudales desaguados.

Perdida en fecha desconocida su función inicial de abastecimiento urbano, se alternaron periodos de abandono y uso de la presa, sin duda con múltiples actuaciones de reparación y modificación de la misma, que solo pasan a estar documentados a partir del final del siglo XVI. Cabe destacar una gran reparación en 1616, así como la instalación de un lavadero de lanas aguas abajo de la presa en 1791, que utilizaba para su proceso el agua embalsada (una descripción de los usos históricos de la presa puede encontrarse en Sereno Martínez, 2002).

En el curso de algunas de estas actuaciones se haría preciso acceder a las galerías de salida de los bocines, que probablemente estarían obstruidas, para aprovechar el agua que podían aportar las mismas, con destino a diversos usos, como molinos (existen varios asociados a la presa), riegos, o el lavado de lanas antes citado. Ello motivó la realización de sendas excavaciones en el espaldón, que crearon una especie de vaguadas en el mismo, debilitando en cierta medida la sección de la presa en estas zonas, y realizándose unos sistemas hidráulicos asociados de cierta complejidad. En el caso del bocín de margen derecha, sabemos que dicha actuación se realizó en 1698, por existir una inscripción en una torre-respiradero que se realizó sobre la galería original romana (Martín Morales, Aranda Gutiérrez y Sánchez Carcaboso, 2001). En el caso del otro bocín no sabemos en que fecha se realiza.

Otra actuación que, en fecha indeterminada, también alteró la sección original del espaldón de la presa fue el trazado sobre el mismo de un camino vecinal, originalmente conocido como "camino de Mérida a Cordobilla" que posteriormente se convertiría en el acceso desde Mérida a la propia presa y a las numerosas edificaciones realizadas en la zona.

En la primera mitad del siglo XX se acomete la puesta en valor de la presa por parte de los Servicios Hidráulicos del Estado, precursores de la Confederación Hidrográfica del Guadiana, organismo que desde su creación en los años 50 se hace cargo de la explotación de la presa. Cabe destacar que esta adscripción de la presa al Estado fue incompleta, por cuanto toda la zona del espaldón de la presa comprendida entre la carretera y el pié del talud quedó en posesión de los propietarios del antiguo lavadero de lanas antes citado.

Entre los años 1990 y 2000 se llevan a cabo sobre la presa actuaciones de gran importancia, pero que sobre todo permitieron aportar datos del máximo interés sobre la misma. Resumiendo mucho, podemos citar que se vació y limpió de fangos el embalse, lo cual permitió descubrir la zona baja de la presa, hasta entonces desconocida, en la cual se encontraron las tomas originales del bocín profundo (dos tuberías de plomo). En las maderas encontradas se realizaron determinaciones del C-14 lo que dio un rango de fechas entre los siglos I y II d.C.

También se realizaron ensayos de C-14 en las muestras de los pólenes hallados en los fangos que rellenaban parcialmente el embalse, resultando para las muestras basales (las del fondo) una edad similar.

Especial importancia tuvieron las diversas campañas de inyecciones realizadas a lo largo de todo el muro, aprovechándose además para dotar a la presa de una extensa red de piezómetros (unos 50) tanto en el muro como, sobre todo, en el espaldón, para controlar el nivel de saturación en el mismo, siendo estos los únicos elementos de auscultación con los que cuenta la presa.

Otra actuación muy relevante para poder realizar una adecuada explotación del embalse,

Figura 2. Presa de Proserpina a embalse vacío, en 1991.

realizada en estas fechas, fue la construcción de un desagüe de medio fondo, constituido por un tubo de 600 mm con dos válvulas de compuerta tipo "Bureau" alojadas en un arquetón situado en un collado lateral en la margen izquierda del embalse.

Todos los datos obtenidos se incorporaron a un estudio titulado "Caracterización histórica, funcional y constructiva del sistema hidráulico de Proserpina" (CHG-Ingeniería 75, 1996).

Finalmente, tras una prologada sequía que mantuvo el embalse vacío varios años, se produjo su llenado rápido en el año 1996, detectándose filtraciones de importancia en la zona del bocín profundo, llegando a ocasionarse la rotura de una de sus paredes. La realización de una intensa campaña de inyecciones localizadas en el entorno de dicho bocín, con testificación geofísica de los resultados, solucionó adecuadamente el problema, procediéndose a la restauración de la pared derrumbada y a la limpieza y acondicionamiento de la galería romana profunda, que estaba completamente obturada por fangos y lechada de cemento, el extremo de la cual se conectó a una red de drenaje que permite evacuar por gravedad las pequeñas filtraciones del bocín y las captadas por la propia galería, causando un efecto muy beneficioso a la presa al deprimir la línea de saturación en la zona (cuando se puso en servicio dicho drenaje, los piezómetros colindantes bajaron unos dos metros de nivel).

Por estas fechas se realiza además otra actuación sobre el embalse consistente en la ejecución de un camino de servicio perimetral, que cumple el doble objetivo de delimitar la zona de Dominio Público Hidráulico y de poner a disposición de los ciudadanos un paseo peatonal a lo largo de todo el embalse (unos 5 km).

2 PROBLEMÁTICA EXISTENTE ANTES DE LAS ÚLTIMAS ACTUACIONES

Las actuaciones antes citadas, que finalizaron hacia el año 2000, dejaron a la presa de Proserpina en unas condiciones de funcionalidad y seguridad adecuadas, aunque seguían existiendo diversos aspectos problemáticos que se consideraba preciso solucionar.

Por un lado estaba el hecho de que una parte importante de la presa, como era todo el talud del espaldón, no fuera de titularidad pública sino privada. En esta zona hay piezómetros, accesos a galerías, respiraderos … elementos a todos los cuales hay que acceder en explotación normal de la presa, además de poder realizar posibles intervenciones de mantenimiento y conservación de los mismos. Y si bien es cierto que los propietarios del antiguo lavadero de lanas nunca pusieron problemas a dichas actuaciones, era una situación no conveniente ni sostenible.

Por otra parte, como se comentó, el entorno de gran parte del embalse está muy urbanizado (de hecho tiene la consideración de zona urbana de Mérida) por lo que el tráfico por la carretera de coronación de la presa tenía cierta intensidad. La carretera y su tráfico no solo suponía un impacto visual negativo sobre la presa, sino que había alterado en cierta medida la sección original del espaldón.

A esto hay que añadir que las dos únicas vías de acceso a la presa de Proserpina, en un caso desde Mérida y en el otro desde la carretera Nacional 630, siguen siendo titularidad de la Confederación Hidrográfica del Guadiana, a pesar de que de hecho se han convertido en vías periurbanas, no estando debidamente acondicionadas para ello.

Además, había otros dos servicios que cruzaban a la margen derecha del embalse aprovechando la coronación de la presa, una línea eléctrica aérea de media tensión y una tubería de abastecimiento de agua, lo que en el primer caso ocasionaba un impacto visual importante, y en el segundo suponía incluso un cierto riesgo para la seguridad de la presa.

Otra problemática era debida a la mínima adecuación de la presa a su visita turística. En efecto, la presa tiene la consideración de Monumento Nacional, y está integrada en el conjunto arqueológico de Mérida, sin embargo las condiciones para su visita turística eran muy precarias, careciendo de facilidades mínimas para los visitantes tales como aparcamientos, rutas peatonales, accesos a los puntos singulares, cartelería informativa …

Finalmente, existían otra serie de pequeñas disfuncionalidades, como dificultades de acceso a ciertos elementos relacionados con la explotación de la presa, el hecho de que ambos bocines estuvieran descubiertos (con los problemas de seguridad y conservación que ello causaba) y no contaran con electricidad, ni por tanto alumbrado.

Para tratar de resolver todas estas circunstancias, se redactaron entre los años 2000 y 2006 los dos Proyectos siguientes:

- Proyecto de ensanche y refuerzo de la carretera de acceso del embalse de Proserpina.
- Proyecto de rehabilitación de la presa y entorno de Proserpina.

El primero de ellos contempla la mejora integral del acceso desde Mérida a la presa, y el segundo, que será sobre el que nos centraremos, una serie de actuaciones encaminadas a dar respuesta a la problemática de la presa y su entorno inmediato antes descrita.

Figura 3. Vista aérea de la presa con la carretera en coronación.

3 EL PROYECTO DE REHABILITACIÓN DE LA PRESA Y ENTORNO DE PROSERPINA

El Proyecto de rehabilitación de la presa y entorno de Proserpina contempla básicamente las actuaciones siguientes:

- Eliminación de la carretera de la coronación de la presa mediante la ejecución de una variante aguas abajo de la misma, con una longitud de unos 1,5 km.
- Eliminación de la tubería de agua de la coronación de la presa, y soterrado de la línea eléctrica de media tensión.
- Construcción de aparcamientos cerca de ambos estribos de la presa.
- Expropiación de todo el espaldón de la presa.
- Reperfilado, mediante pequeños movimientos de tierra, del espaldón de la presa.
- Peatonalización y adecuación para su visita turística de la zona de coronación de la presa.
- Realización en el espaldón de la presa de tres "catas museo" que permiten apreciar la fábrica del muro en la zona de aguas abajo.
- Ordenación y mejora de accesos a elementos relacionados con la explotación de la presa.
- Mejora de la capacidad de desagüe de la obra de fábrica a la que descarga el aliviadero de superficie de la presa.
- Cobertura de las dos torres de toma o "bocines" con unas estructuras que además permiten la visión de su interior.
- Dotación de energía eléctrica y alumbrado a ambos bocines.
- Remodelación de la arqueta de alojamiento de las válvulas del desagüe de medio fondo.
- Sustitución de las cabezas de los piezómetros.

4 EJECUCIÓN DE LAS OBRAS

El 12 de febrero de 2008 se firma el "Convenio de colaboración entre el Ministerio de Medio Ambiente y el Excelentísimo Ayuntamiento de Mérida para la ejecución de infraestructuras viarias en el entorno del embalse de Proserpina y su posterior traspaso al Ayuntamiento".

Dicho Convenio contempla la ejecución de los viales previstos en los dos proyectos anteriormente citados, es decir, la mejora de los accesos a la presa y embalse y urbanizaciones asociadas, recogida en el Proyecto de la carretera, y la construcción de una nueva variante aguas abajo de la presa para liberar la coronación de la misma, recogida en el Proyecto de rehabilitación de la presa.

La firma del mismo impulsa decisivamente la tramitación de ambas obras por parte del Ministerio, saliendo a licitación mediante concurso la obra de la carretera en noviembre de 2008, que fue adjudicada a la empresa Gévora Construcciones, por importe de unos 1,6 millones de Euros.

La obra de rehabilitación de la presa fue incluida en el "Plan E", saliendo a licitación en febrero de 2009, y siendo adjudicada a la U.T.E. Carija-Magenta, por importe de unos 3,2 millones de Euros.

Ambas obras dieron comienzo en el verano del pasado año 2009, estando prevista su finalización en junio de 2010, por lo que a fecha de redacción de esta comunicación (finales de mayo de 2010) se encuentran ya casi terminadas. Es de esperar que se rematen adecuadamente, se reciban y se produzca la cesión de los viales al Ayuntamiento de Mérida tal y como establece el Convenio antes citado.

La obra de la carretera, con unos 3,2 km de longitud, es muy convencional y no ha revestido mayores dificultades, a excepción de la abundancia de restos arqueológicos (de época romana) aparecidos en la zona de ensanche, y las abundantes lluvias

Figura 4. Vista aérea de la variante en construcción.

del Invierno de 2010, todo lo cual ha ralentizado en alguna medida las obras.

El resultado final de las mismas es una carretera de trazado moderno y anchura 7/9 m (calzada/total), a la que además se le adosa una senda ciclable, es decir un carril para uso compartido de peatones y ciclistas. Este carril tiene una anchura de 3 m y se separa de la carretera mediante un bordillo de hormigón.

Las obras de rehabilitación de la presa han resultado más complejas, pero también más interesantes. Se comenzó lógicamente por ejecutar y poner en servicio la variante de la carretera, para poder desviar el tráfico por la misma, lo que fue posible en febrero de 2010. Al igual que en el caso de la carretera, esta fase también fue afectada por hallazgos arqueológicos de la misma época y las intensas lluvias.

Con el tráfico desviado fue posible acometer el grueso de actuaciones sobre la coronación de la presa, comenzando por la demolición de la carretera y los pequeños movimientos de tierra sobre la coronación.

Mención especial merece la excavación de las tres "catas-museo" proyectadas, dos de ellas en los extremos de la presa y una central, de mayores dimensiones, en torno al punto donde se unen las dos principales alineaciones rectas de la presa formando un ángulo obtuso. Estas excavaciones se realizaron mediante métodos arqueológicos, habiendo sido subcontratadas al Consorcio de la Ciudad Monumental, Histórico-Artística y Arqueológica de Mérida, que también ha realizado el seguimiento arqueológico general de ambas obras.

Una vez excavadas las catas se acondicionan para su contemplación mediante el empleo de empalizadas de madera y relleno de gravilla sobre geotextil en el fondo, y la creación de unas plataformas peatonales, formadas por traviesas antiguas de ferrocarril y protegidas con barandillas. La cata central ha quedado muy vistosa, permitiendo apreciar los contrafuertes existentes hacia aguas abajo, cuya existencia era conocida aunque no eran visibles por estar embebidos en el espaldón bajo el nivel de coronación.

Figura 5. Cata-museo central apreciándose los contrafuertes.

Figura 6. La conducción romana en la zona donde empieza a aparecer con su sección casi completamente conservada.

Procede destacar que las intensas lluvias aludidas, ocasionaron que el nivel del embalse subiera por encima del fondo de las excavaciones de las "catas-museo", no apreciándose más que pequeñas filtraciones por el muro, que eran absorbidas sin problemas por el fondo de la excavación (al igual que el propio agua de lluvia) sin que se llegaran a formar charcos, confirmando así la buena capacidad drenante del espaldón, lo que es muy conveniente para la presa.

Los movimientos de tierra sobre la coronación y el espaldón se han limitado a rellenos localizados en determinadas zonas, en particular donde la carretera iba en cierto desmonte, o en las vaguadas artificiales creadas por excavación para acceso a las galerías de salida de ambos bocines.

En la salida del bocín de margen izquierda, el menos profundo, se estimaba que debía estar el origen del acueducto romano hasta Mérida, debido a que la cota de la galería de salida del mismo es coherente con la de los diversos restos de dicha conducción, que con un trazado de unos 9 km llegaba hasta la ciudad, entrando en la misma a través del monumental acueducto de Los Milagros.

Por tanto, previamente a efectuar otras actuaciones en esta zona, se realizó una prospección y posterior excavación arqueológica, cuyos resultados fueron auténticamente espectaculares, pues se encontraron restos de la conducción romana prácticamente a la salida actual de la galería de salida del bocín, y a la misma cota que la misma, lo cual nos indica sin género de dudas que existía una conexión hidráulica entre la conducción y la presa,

confirmando el origen romano de ésta y su función de "*caput aquae*" del acueducto.

Por desgracia, la excavación realizada en su día para acceder a la galería con el fin aprovechar las aguas susceptibles de ser aportadas por la misma (para molinos, riegos, lavado de lanas …), debió destruir casi completamente el tramo de conducción más próximo al actual final de la galería, de forma que los restos encontrados corresponden tan solo a la cimentación de la conducción. Sin embargo algo más adelante, a unos 18 m de la salida de la galería, sí encontramos ya la conducción bastante íntegra, a excepción de la bóveda que la cubría, conservándose incluso el revestimiento de "*opus signinum*". El tratamiento definitivo de estos restos está pendiente de definición, a consensuar con el Consorcio.

Como se comentó anteriormente, el acondicionamiento de la presa para su visita turística era otro de los objetivos principales de la actuación. Para ello, se han realizado dos aparcamientos próximos a ambos estribos de la presa, desde los cuales se puede acceder peatonalmente a la misma, el del estribo derecho es pequeño, con capacidad de unos cinco vehículos tipo turismo, pero el del estribo izquierdo es mayor, con capacidad para unos quince turismos y dos autobuses.

Toda la zona de la coronación de la presa queda de acceso peatonal libre, realizándose una red de caminos con pavimento de losas de granito sobre una base de hormigón, constituida por un camino principal de 3 m de anchura, que recorre la coronación de estribo a estribo, y una serie de caminos auxiliares, de 1,5 m de anchura (en algún caso con escaleras para salvar desniveles), que dan acceso desde el principal a las catas-museo y bocines. En los recintos delimitados por estos caminos más alejados del muro de presa se han realizado plantaciones de especies arbustivas.

La zona del talud del espaldón, por motivos de seguridad, queda de acceso restringido, para asegurar lo cual se ha realizado un cerramiento constituido por un zócalo de hormigón chapado en granito sobre el que se coloca una celosía de metal trenzado, consiguiendo un efecto estético adecuado y evocador en alguna medida de los enrejados de época romana. En este cerramiento se dejan dos puertas que dan acceso a unas escaleras, también chapadas en granito, que permiten un cómodo acceso al espaldón de la presa, hacia ambas salidas de las galerías de los bocines, zonas a las que periódicamente es preciso acceder dentro de la explotación normal de la presa.

A los dos bocines se les dota de una cobertura, precisa por motivos de seguridad y conservación, formada por una estructura de madera en forma de tejadillo a dos aguas, con uno de los lados constituido en su mayor parte por un vidrio de seguridad de forma que permita la iluminación y contemplación del interior del bocín, lo que puede realizarse desde una plataforma ubicada delante del vidrio, a la que se accede mediante unas escaleras. El resto de la estructura se reviste con chapa de cobre, que una vez oxidada tiene un aspecto adecuado a la monumentalidad del entorno.

Además, se les dota de energía eléctrica e iluminación, y se realizan algunas mejoras en sus puertas de acceso y barandillas de sus escaleras interiores.

Está prevista la colocación de cartelería informativa en los puntos de mayor interés, como son los dos bocines, las tres catas museo y un mirador también realizado desde el que se contempla la salida de la galería del bocín de margen izquierda y los primeros restos de la conducción romana.

Finalmente, se han realizado también actuaciones específicas para mejorar la seguridad y funcionalidad de la presa y facilitar su explotación. La obra de fábrica existente a la salida del aliviadero

Figura 7. Red de caminos peatonales en coronación de la presa.

Figura 8. Estructura de cobertura de uno de los bocines.

de superficie de la presa (conocido como "la sangradera") se ha ampliado para permitir el paso de la avenida de 1.000 años. Igualmente, se ha modificado la cobertura del arquetón que aloja las válvulas del desagüe de medio fondo, facilitando el acceso al mismo creando sobre el mismo una caseta que se remata de forma que recuerde en cierta medida al tratamiento dado a los bocines, antes comentado, ya que conceptualmente se trata de una obra con la misma finalidad que éstos. También se sustituyen las cabezas de los piezómetros.

5 CONCLUSIÓN

A fecha de redacción de esta Comunicación (finales de mayo de 2010) las obras en realización sobre la presa de Proserpina, su entorno inmediato y sus accesos, están muy avanzadas, estimándose que salvo algún remate, puedan estar completamente finalizadas para mediados del mes próximo.

Estas obras no son sino un elemento más en un proceso histórico de casi veinte siglos, pero entendemos que con las mismas se ha conseguido realizar una rehabilitación estética y funcional de la presa de enorme importancia, teniendo en cuenta que actualmente el uso principal de la presa y su embalse es el turístico-recreativo.

En este sentido cabe señalar además que el Ayuntamiento de Mérida también acaba de realizar sobre el embalse unas obras de acondicionamiento del mismo para ordenar sus usos y mejorar su aprovechamiento recreativo.

Cabe destacar también los interesantísimos descubrimientos arqueológicos realizados a consecuencia de estas obras, entre los que destaca, como se comentó, el haber conseguido establecer sin ningún género de dudas la conexión hidráulica entre la presa y el acueducto romano.

Sin embargo no conviene olvidar que aun quedan importantes asignaturas pendientes en relación con el embalse de Proserpina y su entorno inmediato, siendo sin duda alguna la fundamental la necesidad de dotar a las urbanizaciones colindantes de un adecuado sistema de saneamiento y depuración de sus aguas residuales, actualmente muy inadecuado, lo que tendría efectos muy beneficiosos sobre la calidad de las aguas del embalse.

Es de esperar que las diversas Administraciones implicadas seamos capaces de coordinar esfuerzos para conseguir que esta verdadera joya de la ingeniería de todos los tiempos que es la presa de Proserpina resplandezca en un entorno ambientalmente adecuado. Pensamos que el camino está marcado, pero hay que seguir recorriéndolo.

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Inyección de filtraciones, acondicionamiento del drenaje y control de subpresiones en la presa de Portbou (Girona)

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ABSTRACT: Concrete dams can present leakage through construction joints. This can lead to instability due to uplift pressures and reduction of cohesion. Water flowing through cold joints can cause damage: dissolution and washing of the cement, acceleration of the expansive reactions in concrete, etc. The aesthetic effect is, also, very negative. We describe a procedure for injection of cold joints applied to Portbou's dam (Girona) with excellent results. High viscosity epoxy resins were pumped at very high pressures.

1 INTRODUCCIÓN

Las filtraciones por juntas frías de hormigonado son defectos típicos en presas de hormigón vibrado asociado al procedimiento constructivo o calidad de hormigón de la presa.

Las juntas frías permeables constituyen zonas de debilidad de la presa ya que suponen la existencia de cierta subpresión en toda o parte de la junta y colaboran al posible deslizamiento de ésta. De igual forma una pantalla de drenaje eficaz en una presa es un elemento importante para garantizar su estabilidad al deslizamiento frente a la acción del nivel de agua.

La presa se ubica en un enclave de difícil acceso. Este aspecto, añadido a las reducidas dimensiones de la infraestructura, supuso un factor añadido de dificultad en fase de ejecución de obras que se tradujo en una merma en la calidad del hormigón y un limitado cuidado en la ejecución del tratamiento de superficie entre tongadas (limpieza y retoma).

En el año 2009 la Agencia Catalana del Agua (ACA) decidió realizar el sellado de las principales filtraciones mediante inyección a alta presión de resinas epoxi de elevada viscosidad. Además se instaló una red de piezómetros para auscultación de subpresión y finalmente se ejecutó una nueva pantalla de drenaje. La obra fue ejecutada por la empresa HCC.

En el texto se describen los trabajos realizados y los resultados y conclusiones obtenidas.

2 DESCRIPCIÓN GENERAL DE LA PRESA DE PORTBOU

La presa de Portbou, cuya construcción concluyó en 1.974, está situada en la riera de Portbou (Girona). Es de tipo gravedad y planta ligeramente curva.

La altura total desde cimientos es 28 m. y la longitud de coronación 94 m. Los taludes son 0,16 aguas arriba y 0,65 aguas abajo.

Figura 1. Foto aérea de la presa de Portbou.

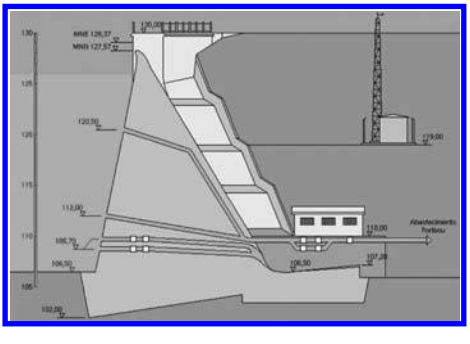

Figura 2. Esquema transversal.

3 DESCRIPCIÓN DE FILTRACIONES

La presa de Portbou presentaba, antes de las obras que aquí se describen, varias filtraciones por juntas frías de hormigonado, claramente visibles en el paramento de aguas abajo de la presa.

3.1 *Causas de la aparición de filtraciones*

Las filtraciones que se producen a través de las juntas frías de hormigonado es un defecto típico en presas de gravedad de hormigón vibrado debido al propio procedimiento constructivo de la presa.

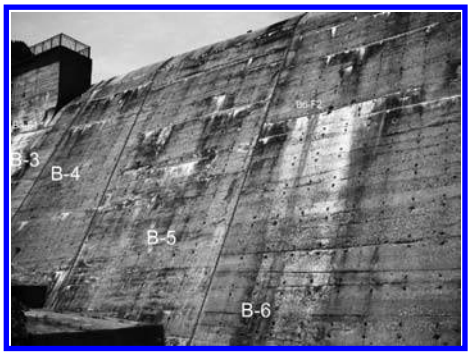

Figura 3. Filtraciones por juntas frías en bloques 3, 4, 5 y 6.

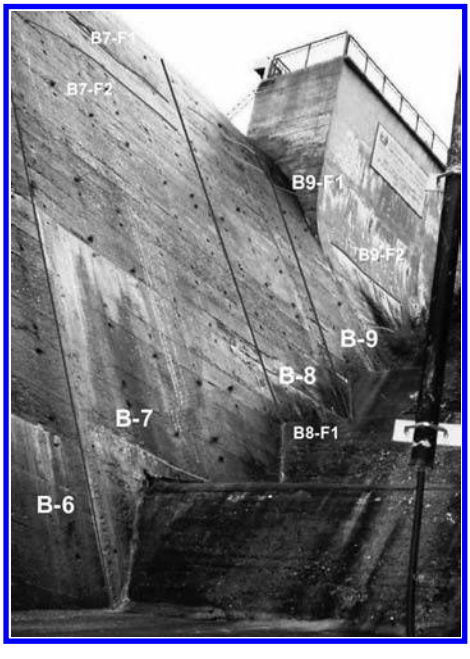

Figura 4. Filtraciones por juntas frías en bloques 6, 7, 8 y 9.

Figura 5. Obras de construcción de la presa de Portbou (1967).

Existen distintos tipos de tratamientos para asegurar la buena unión entre tongadas sucesivas pero, indudablemente siempre pueden verse afectados por el propio desarrollo de la obra y por las dificultades en fase de ejecución, por lo que constituyen una superficie de posible debilidad y de aparición de filtraciones.

En el caso de la presa de Portbou, la calidad de ejecución del propio hormigón de cuerpo de presa, así como de la unión entre distintas tongadas, fue bastante deficiente y es la causa de las filtraciones a través del cuerpo de presa que se manifiestan en el paramento de aguas abajo.

El reducido volumen de hormigón (13.200 m^3) necesario para la ejecución del cuerpo de presa no justificó la instalación en obra de una central de hormigonado que asegurara la fabricación del hormigón al ritmo necesario. Por otra parte, el camino de acceso a la presa no es apto para la circulación de cierta tipología de vehículos/maquinaria de obra, sobre todo para camiones, hormigoneras, ... lo que también alteró el normal ritmo de hormigonado de tongadas de bloques.

4 SELLADO DE JUNTAS FRÍAS

Se ejecutaron los siguientes trabajos:

- Diseño de la red de taladros de inyección
- Replanteo taquimétrico de juntas frías en paramento aguas abajo de la presa
- Montaje de andamio en paramento de aguas abajo
- Perforación de taladros de inyección
- Pruebas de agua
- Inyección a alta presión de resinas epoxi acuaestables.

4.1 *Diseño de la red de taladros de inyección*

Para diseñar la red de taladros de inyección existían dos posibilidades (Fig. 6):

– Perforaciones desde galería de inspección de la presa
– Perforaciones desde el paramento de aguas abajo sobre andamio

Para decidir qué opción era más ventajosa se realizó un análisis económico, técnico y de variación de plazos.

Económicamente las perforaciones desde galería ofrecían la ventaja de ahorrar el coste de la instalación de andamios pero, por el contrario, las longitudes de perforación eran mayores.

Para poder estudiar los costes con exactitud y conocer las longitudes de perforación necesarias en caso de perforar desde galería, se realizó un levantamiento taquimétrico de las juntas frías húmedas y del resto de la presa (Fig. 7).

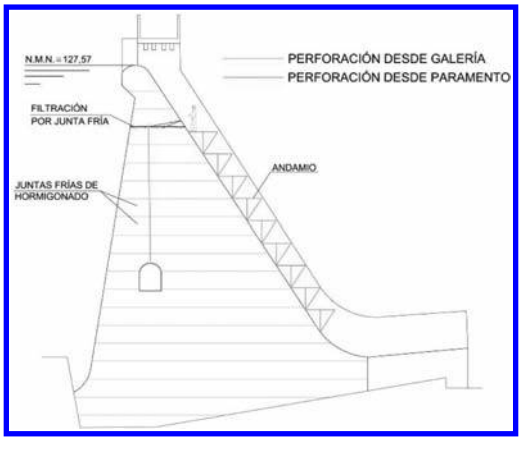

Figura 6. Red de taladros de inyección. Alternativas de diseño.

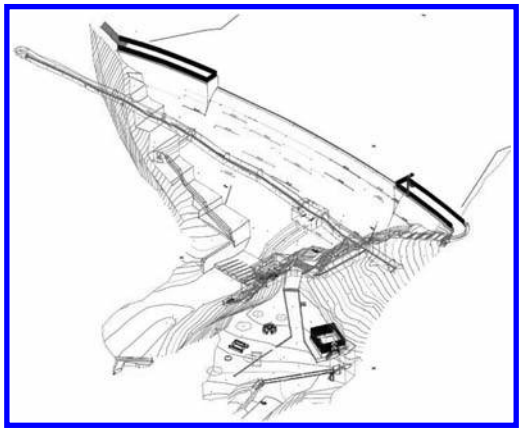

Figura 7. Levantamiento topográfico en 3D de la presa.

El resultado del estudio de alternativas resultó favorable a la instalación de andamios en todos los aspectos analizados.

Por otra parte, técnicamente esta opción era ventajosa por otros motivos: las perforaciones desde galería implicarían atravesar otras juntas frías lo que obligaría a realizar obturaciones en profundidad para evitar que la resina pudiera fluir por juntas que no se preveía tratar y al poder acceder al propio labio de fisura permitía el control de movimientos de los labios de la fisura durante la inyección.

4.2 *Replanteo taquimétrico de juntas frías*

Se realizó un levantamiento taquimétrico en 3D del paramento de aguas abajo de la presa, con la ubicación exacta de las juntas frías que presentaban filtraciones, y del propio cauce, incluyendo las laderas aguas abajo de la presa.

4.3 *Montaje de andamio en paramento de aguas abajo*

Se montó un andamio LAYHER de estructura tubular metálica del tipo ALLROUND anclado al paramento de aguas abajo y apoyado en la solera del cuenco amortiguador del aliviadero.

Figura 8. Montaje de andamio en paramento de aguas abajo.

4.4 *Perforación de taladros de inyección*

La red de taladros de inyección se ejecutó a rotación con corona de diamante. La perforación a rotación presenta importantes ventajas técnicas respecto la perforación a rotopercusión: permite la recuperación de testigo inalterado, apenas introduce tensiones en el medio a inyectar, se evitan desvíos en la trayectoria, etc.

Se utilizaron equipos de perforación versátiles y ligeros que permitiesen transportarlos por diferentes niveles de los andamios.

El hecho de perforar desde andamios permitió emboquillar los taladros 60 cm por encima de la junta fría a inyectar. Con ello se consigue que todo el taladro de inyección esté en la misma tongada de hormigonado, evitándose así que atraviese otras juntas frías lo que complicaría la inyección y supondría ineficiente de resina.

Durante la perforación, en muchos casos, se pudo comprobar la discontinuidad correspondiente a la intersección con la junta fría correspondiente.

4.5 *Pruebas de agua*

Previamente a la inyección se realizaron las correspondientes pruebas de agua.

Las pruebas de agua tienen una importancia fundamental, ya que permiten conocer las comunicaciones entre taladros, la extensión de la región a inyectar y la posible presencia de coqueras y fugas. De esta forma es posible diseñar la metodología más apropiada de inyección, establecer las secuencias entre taladros, determinar los puntos de fuga a controlar, etc. En la Fig. 9 se puede comprobar cómo la inyección de agua por los taladros de aguas abajo de la junta B9-F2 comunica con el labio de la fisura en el paramento de aguas arriba, ratificando que la junta es una vía franca de paso de agua y que la red de taladros de inyección intersecta adecuadamente dicha superficie.

Figura 9. Perforación de taladros de inyección (junta B7-F1).

Figura 10. Testigo recuperado con intersección con junta fría.

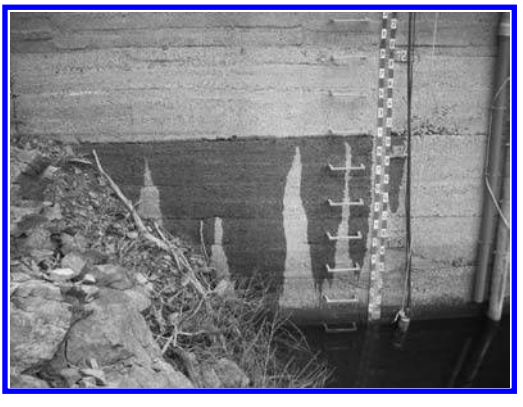

Figura 11. Prueba de agua en junta B9-F2 (aguas arriba).

4.6 *Sellado de filtraciones mediante inyección de resinas*

La obra se ejecutó con el embalse prácticamente lleno ya que el vaciado del mismo era inviable por motivos medioambientales y porque su principal uso es el abastecimiento del núcleo urbano de Portbou.

Todos estos condicionantes obligaban al uso de productos de inyección no contaminantes ni miscibles en agua, descartando obviamente, la lechada de cemento.

Los productos utilizados para la inyección fueron resinas epoxi acuaestables de diferentes viscosidades, cuya designación es:

- HCC-AV
- HCC-MV
- HCC-BV

La inyección con resinas epoxi logra un doble objetivo:

– Rellenar los huecos existentes en la fisura impermeabilizándola
– Recuperar el monolitismo de la estructura

La recuperación del monolitismo estructural se consigue gracias a las elevadas características resistentes de las resinas inyectadas una vez fraguadas. En la Fig. 12 se puede apreciar el testigo recuperado de la intersección de un taladro de comprobación con una junta inyectada con resina (color verde).

Los ensayos de rotura a compresión de probetas que interceptan una junta inyectada rompen en un 99% de veces por el hormigón.

Para la inyección de resinas se utilizó una inyectadora electro-hidráulica capaz de inyectar hasta 600 bar de presión. La inyectadora a emplear debió ser tal que permitiera realizar trabajos en zonas de difícil accesibilidad como andamios, etc; con circuito de aceite separado del cuerpo de inyección, de manera que se facilitara su instalación en los andamios.

Se colocaron relojes comparadores en el labio de la fisura a inyectar para control de movimientos durante la inyección. La baja carga hidráulica durante la fase de inyección (3,80 m de carga sobre la fisura ubicada a menor cota) fue un factor a favor de la seguridad de los trabajos. Aún así se pudo comprobar que los movimientos verticales fueron pequeños, del orden de varias centésimas de milímetro.

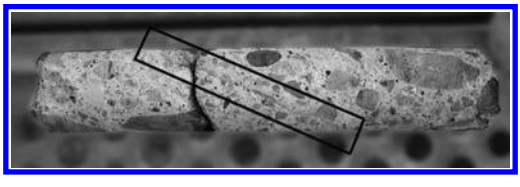

Figura 12. Taladro de comprobación después de la inyección.

Figura 13. Inyección junta B9-F2 (aguas abajo).

Figura 14. Inyección de juntas B7-F1 y B7-F2 (aguas arriba).

Durante la inyección se pudo comprobar como la resina comunicaba con todo el labio de fisura asegurando que se rellenaba con resina toda la discontinuidad garantizando su impermeabilidad y el monolitismo estructural (ver Figs. 13 y 14).

5 INSTALACIÓN DE RED DE PIEZOMETRÍA

Además de los trabajos de inyección, se ejecutaron trabajos para disponer de una red completa de sensores de piezometría.

Esta red permitirá obtener información vital en relación a la ley de subpresiones en el plano de contacto presa-cimiento de cara a controlar su evolución y analizar la estabilidad al deslizamiento de la presa.

La red de piezometría ejecutada consta de un total de once piezómetros de cuerda vibrante. Se ha colocado 1 piezómetro por bloque exceptuando el bloque central de la presa (Nº 6) donde se han colocado dos, que interceptan al plano de contacto en diferentes puntos para la misma sección transversal.

Las perforaciones de los taladros (ver Fig. 15) se inclinaron lo necesario (α) de forma que la ubicación del piezómetro en el plano de contacto quedase ligeramente aguas abajo del punto de intersección, con ese mismo plano, de la red de drenaje ($D_{dren\text{-}piez,\ r}$). Las razones fueron no generar una línea de perforaciones no al tresbolillo que pudiera suponer un plano preferencial de fracturación en la base de la presa.

6 REHABILITACIÓN DE LA RED DE DRENAJE

La red de drenaje que se ejecutó en fase de construcción de la presa ha ido perdiendo eficacia

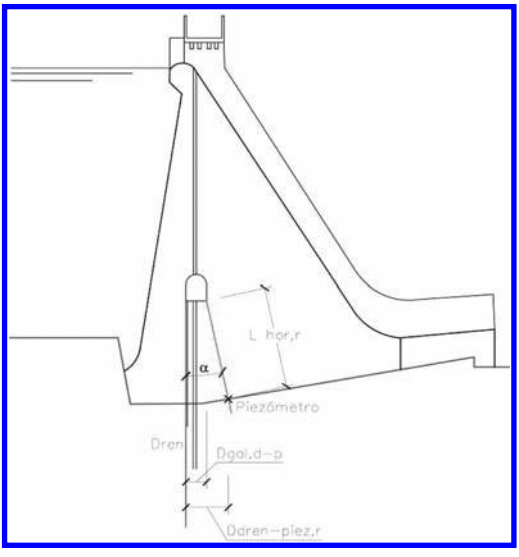

Figura 15. Perfil transversal—ubicación piezómetros.

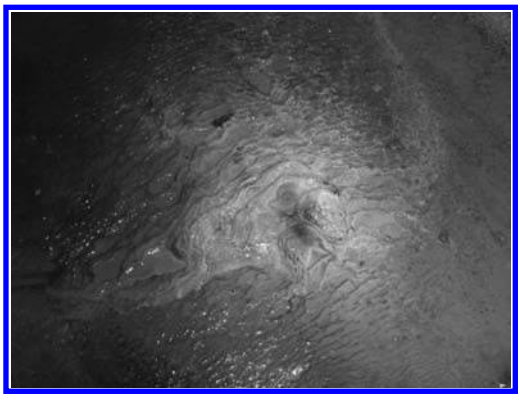

Figura 16. Boca de dren totalmente obstruido.

hasta el punto de presentar un importante grado de obstrucción en la gran mayoría de los drenes que la conforman (Fig. 16). Hay que destacar que el estado de obstrucción de esa red de origen no aconsejó la rehabilitación de drenes por lo que se procedió a ejecutar nuevas perforaciones al tresbolillo con las existentes para evitar la formación de un plano preferencial de posible fracturación.

Tanto los taladros asociados a trabajos de inyecciones como los correspondientes a la implantación de la red de control piezométrico se ejecutaron antes que la nueva red de drenaje. El objetivo principal de este orden lógico de ejecución era el disponer de unas lecturas del estado inicial de partida de subpresiones y de filtraciones no alteradas por el futuro drenaje del plano de contacto. De esta forma, con estos datos de referencia y a medida que se iban ejecutando nuevos drenes, fue posible analizar la eficiencia de los trabajos por medio de la variación del estado piezométrico (reducción de subpresión) y de caudales aforados (aumento de caudal por nuevos drenes). El análisis continuo del estado piezométrico en el plano de contacto durante la ejecución de taladros permitió optimizar la longitud total de la nueva red de drenaje. Aún así, y para el caso que tratamos, el no disponer de un nivel máximo de embalse, el propio diseño de de la galería de inspección y la aparente impermeabilidad de la cerrada supusieron ciertas limitaciones a una optimización máxima de los trabajos.

7 LIMITACIONES AL DRENAJE ÓPTIMO

El poder optimizar de longitud total de drenes a ejecutar en la rehabilitación del drenaje de una presa depende en gran medida de:

- Disponer de un nivel de embalse máximo sostenido (límite superior de compuertas o umbral en aliviadero de labio fijo) para asegurar el poder trabajar con una la red de corriente de mayor solicitación en cimiento y contacto.
- Disponer de información del cimiento a una profundidad razonable (mín. 1/3 de la carga máxima hidrostática para la sección de estudio) en referencia a vías preferentes de circulación de agua (diaclasado, fracturas, estratificación, etc. …) para optimizar la profundidad de taladros a ejecutar.
- Existencia de una red de piezómetros que mida subpresiones en el plano de contacto presa-cimiento para diseñar de forma óptima el interespaciado de drenes.

En el caso de la presa de Portbou, el nivel de embalse en el momento de ejecución de trabajos se situó, de media, 5,60 m por debajo del umbral de aliviadero de labio fijo. Esto supuso disponer en cimiento del 78% (20,00 m.c.a.) de la carga hidrostática máxima posible (25,57 m.c.a.). Este hecho no afectó a la hora de diseñar la profundidad de los drenes a ejecutar.

De cara a disponer de información del cimiento a una profundidad de 10 metros por debajo del plano de contacto se ejecutaron unos ensayos geofísicos de investigación.

De forma simplificada, estos ensayos (rojo en la Fig. 18) consisten en la ejecución de unos taladros por los que se introducirá una sonda especial "multiparamétrica" (caudal circulante, temperatura y conductividad) que permitirá identificar aquellas cotas por debajo del cimiento en que se producen aportaciones de agua significativas. Dichas aportaciones son detectadas en los

gráficos de registro generados (Fig. 17) al presentarse discontinuidades en cuanto a algunos de los tres parámetros de medición. Lamentablemente para el caso de la presa de Portbou, bien sea por la limitada carga hidrostática de trabajo, por tratamientos realizados en el cimiento en origen o por la propia impermeabilidad del cimiento, estos trabajos no aportaron información adicional que fuera de utilidad a la hora de plantear el diseño de la red de drenaje en cuanto a su profundidad se refiere. Por esta razón se ejecutaron drenes (azul en la Fig. 18) de longitud típica teórica igual al tercio de la carga máxima de agua en una sección de trabajo.

Así pues tan sólo se pudo recurrir a la red piezométrica para intentar minimizar la longitud final de drenes optimizando el interespaciado entre taladros. Una vez instalados los sensores (verde en la Fig. 18) comenzó la toma de datos de subpresión para análisis continuo de su evolución durante la ejecución de taladros dren. Para cada bloque de la presa, las perforaciones se plantearon inicialmente a una distancia determinada que permitiera ir intercalando nuevos drenes si así lo aconsejaba el estado de subpresiones.

La ley de subpresiones de referencia tomada como máxima admisible fue la triangular teórica reducida un tercio en la vertical coincidente con la red de drenaje (Fig. 19). Los valores reales medidos con los piezómetros instalados se contrastaban contra esta referencia para analizar si la superaban y/o cuánto iban reduciéndose durante la ejecución de taladros dren.

En seguida se observó, con los primeros drenes ejecutados en cada uno de los bloques de la presa, un aspecto de interés que impediría, en parte, optimizar la longitud total de red de drenaje en cuanto a interespaciado se refiere. Un dato de partida a conocer antes de la ejecución de un dren es la distancia entre la boca de éste y el plano de contacto presa-cimiento (valor P en Fig. 9 y gráfica en rojo en Fig. 20). Esta distancia determina una subpresión característica de vital importancia: para un bloque concreto, como por ejemplo el de la Fig. 19, los valores de subpresión superiores a la longitud de sus drenes entre el plano de contacto y solera de galería donde liberan presión (magnitud P) nunca podrán ser reducidos, por medio de una red de drenaje convencional, a un valor menor que esa columna de agua (P).

Trasladando este concepto a la presa de Portbou, en la Fig. 20 graficamos conjuntamente en un alzado de la presa la información correspondiente a:

– Ley de subpresiones máxima admisible de referencia (área sombreada en azul) atendiendo al nivel de embalse en la época de ejecución de las obras.

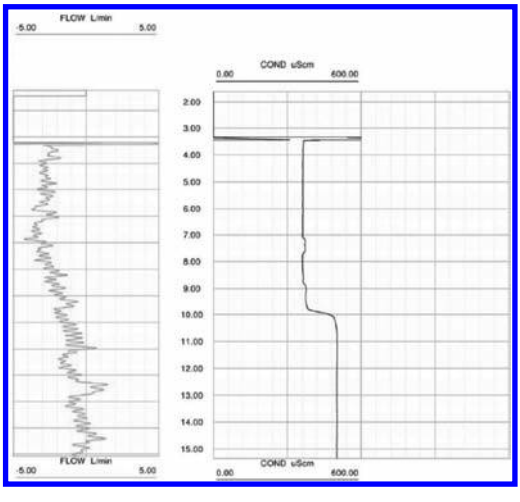

Figura 17. Registros de ensayo. Aportación de caudal a 9 m por debajo de boca de taladro.

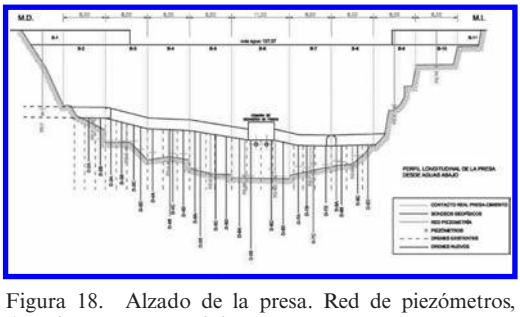

Figura 18. Alzado de la presa. Red de piezómetros, drenaje y ensayos geofísicos.

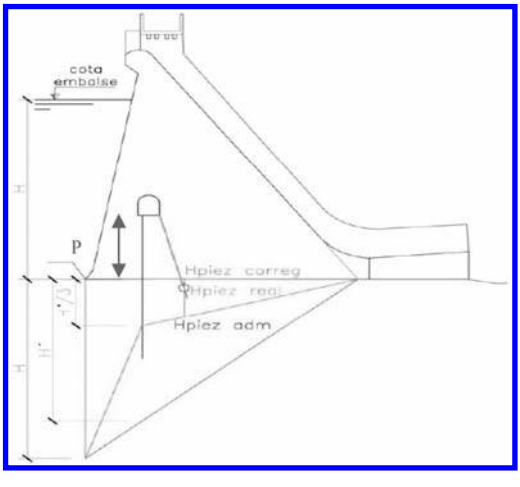

Figura 19. Esquema de magnitudes consideradas para el cálculo de subpresiones máximas de referencia.

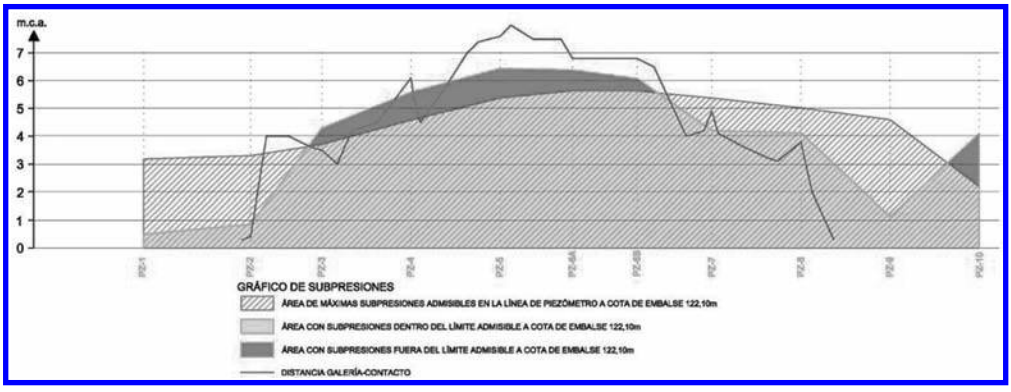

Figura 20. Gráfico de representación para nivel a umbral de aliviadero de subpresiones máximas de referencia, subpresiones medidas y distancia plano de contacto-boca de dren en galería.

– Ley de subpresiones reales medidas: En verde se indican las zonas con valores inferiores a los máximos teóricos admisibles y en rojo se sombrean aquellas zonas en que se superan esos límites de referencia.
– Para cada dren, distancia entre solera de galería y plano de contacto presa-cimiento (gráfica en rojo): en caso de existencia de subpresiones (m.c.a.) mayores a esas distancias (m.c.a.) sólo serán reducidas a un valor equivalente a la distancia galería-plano contacto.

Del análisis conjunto de esta información, rápidamente se puede observar que la zona central de la presa presenta subpresiones mayores a las teóricas admisibles pero menores que la distancia entre bocas de drenes (solera de galería) y plano de contacto por lo que por medio de una red de drenaje convencional no se conseguirá reducir el estado de subpresiones máximo que se pueda presentar.

Posterior a la finalización de los trabajos, episodios de lluvias se tradujeron en un aumento del nivel de embalse hasta lograr verter por aliviadero y mantenerse esa cota durante un tiempo. En todo el proceso se tomaron lecturas de los piezómetros instalados y de los caudales de filtración (Fig. 21).

Con este evolutivo se pudo apreciar que el aumento de carga hidrostática en 8 metros apenas supuso un aumento de subpresiones en 0,50 m.c.a. y que tan sólo tres drenes incrementaron muy levemente (1 l/min) el caudal de alivio situándose el caudal total máximo drenado en valores muy reducidos de 12 l/min (Fig. 21). Estos resultados son coherentes con aquello comentado con respecto a la Figura 20 y también dan idea de que el cimiento y del contacto presa-cimiento presentan un grado de impermeabilidad satisfactorio.

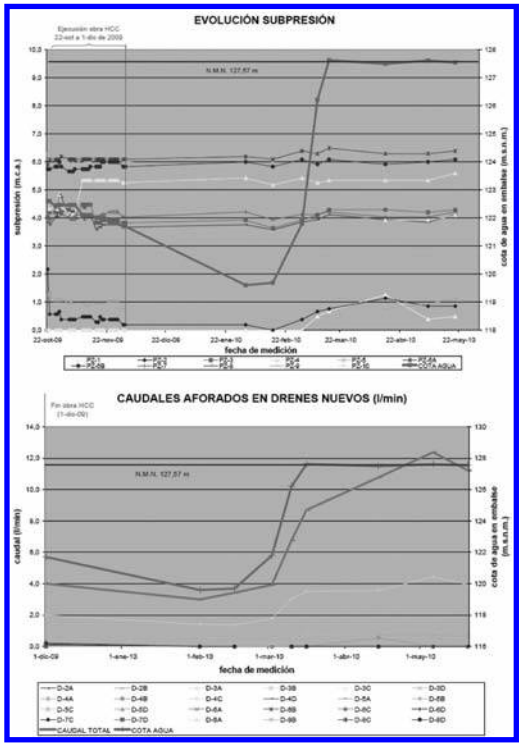

Figura 21. Evolución de subpresiones y caudales de filtración.

Ante esta situación cabe replantearse el cómo proceder a drenar la superficie del contacto por medio de otras vías o el analizar la estabilidad de la presa frente a deslizamiento considerando ese estado de fuerzas inestabilizantes detectado. Para el caso de la presa de Portbou se decidió realizar un cálculo al deslizamiento para comprobar si se daba

cumplimiento y con qué grado a aquello exigido por la normativa actual y recomendaciones técnicas específicas.

8 CÁLCULO AL DESLIZAMIENTO

A pesar del aligeramiento que existe en los bloques centrales de la presa debido a la presencia del aliviadero de labio fijo y ausencia de carretera de coronación, dado que la suma de taludes es de 0,65 + 0,18 = 0,81, antes incluso de realizar el cálculo de estabilidad al deslizamiento, lo más probable es que se obtengan coeficientes de seguridad que cumplan las exigidas o recomendadas en la *Instrucción para el proyecto, construcción y explotación de grandes presas* o Guías Tècnicas de Seguridad de Presas respectivamente.

Para la realización de los cálculos se ha considerado una sección tipo cimentada en una contrapendiente de un 15% similar a lo que se ejecutó en obra (Fig. 22). Este planteamiento queda del lado de la seguridad pues se ha despreciado el efecto de un rastrillo ejecutado en pié de aguas arriba.

Los valores de rozamiento y cohesión considerados también han sido conservadores para las pizarras presentes en cimiento:

– Cohesión (C): 40 tn/m^2
– Ángulo de rozamiento (Ø): 30º

Con estas hipótesis se realizan los cálculos oportunos obteniendo, para todas las solicitaciones de

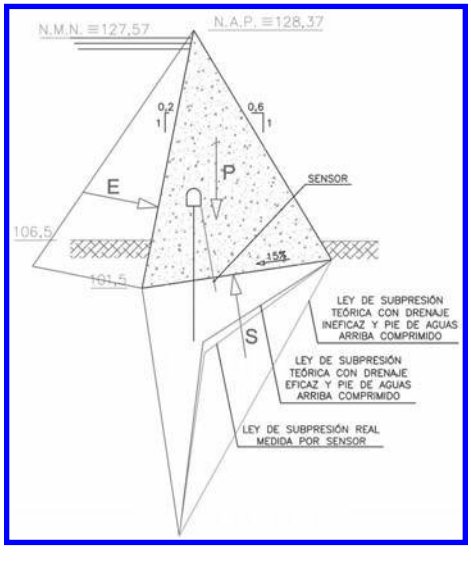

Figura 22. Sección transversal y fuerzas consideradas para el cálculo de estabilidad.

VALORES DE LAS CARGAS A CONSIDERAR

CARGA	COD.	VALOR (TN)
PESO PROPIO	P	591,11
	P_N	584,57
	P_T	87,68
EMPUJE HIDROSTÁTICO	MÁXIMO NIVEL NORMAL DEL EMBALSE (127,57 m)	
	E	346,55
	E_H	339,82
	$E_{H,T}$	336,06
	$E_{H,N}$	50,41
	E_V	66,64
	$E_{V,T}$	9,89
	$E_{V,N}$	65,91
	AVENIDA DE 500 AÑOS (128,37 m)	
	E	368,14
	E_H	361,00
	$E_{H,T}$	357,00
	$E_{H,N}$	53,55
	E_V	70,80
	$E_{V,T}$	10,50
	$E_{V,N}$	70,01
SUBPRESIÓN	NIVEL NORMAL DEL EMBALSE (127,57 m)	
	$S_{NMN,E}$ Drenes eficaces	135,60
	$S_{NMN,I}$ Drenes ineficaces	259,94
	$S_{NMN,R}$ Subpresiones reales	146,67

CÁLCULOS DE ESTABILIDAD AL DESLIZAMIENTO

COMBINACIÓN SOLICITACIONES		CONDICIÓN ESTABILIDAD	COEFICIENTES DE SEGURIDAD			
			EXIGIDOS		REALES	
COD.	DESCRIPCIÓN		F_1	F_2	F_1	F_2
A2	N.M.N. (127,57 M) + DRENES EFICACES	$E_{e,l} \le E_{r,l} + P_l + \frac{(P_n + E_{e,n} + E_{r,n} - S)\tan\phi}{F_1} + \frac{C \cdot A}{F_1}$ $336{,}06 \le 9{,}89 + 87{,}68 + \frac{(584{,}57 + 50{,}41 + 65{,}91 - 135{,}6)\tan(30)}{F_1} + \frac{40 \cdot 19{,}94}{F_2}$ $336{,}06 \le 9{,}89 + 87{,}68 + 217{,}58 + 159{,}52$ $336{,}06 \le 474{,}67$	1,5	5	2,4	7,9
A2*	N.M.N. (127,57 M) + SUBPRESIONES REALES (MEDIDAS POR PIEZÓMETROS)	$E_{e,l} \le E_{r,l} + P_l + \frac{(P_n + E_{e,n} + E_{r,n} - S)\tan\phi}{F_1} + \frac{C \cdot A}{F_1}$ $336{,}06 \le 9{,}89 + 87{,}68 + \frac{(584{,}57 + 50{,}41 + 65{,}91 - 146{,}67)\tan(30)}{F_1} + \frac{40 \cdot 19{,}94}{F_2}$ $336{,}06 \le 9{,}89 + 87{,}68 + 213{,}32 + 159{,}52$ $336{,}06 \le 470{,}40$	1,5*	5*	2,3	7,8
B21	N.M.N. (127,57 M) + DRENES INEFICACES	$E_{e,l} \le E_{r,l} + P_l + \frac{(P_n + E_{e,n} + E_{r,n} - S)\tan\phi}{F_1} + \frac{C \cdot A}{F_1}$ $336{,}06 \le 9{,}89 + 87{,}68 + \frac{(584{,}57 + 50{,}41 + 65{,}91 - 259{,}94)\tan(30)}{F_1} + \frac{40 \cdot 19{,}94}{F_2}$ $336{,}06 \le 9{,}89 + 87{,}68 + 212{,}15 + 199{,}40$ $336{,}06 \le 509{,}12$	1,2	4	2,1	6,9
B23	N.A.P. 500 AÑOS (128,37 M) + DRENES EFICACES	$E_{e,l} \le E_{r,l} + P_l + \frac{(P_n + E_{e,n} + E_{r,n} - S)\tan\phi}{F_1} + \frac{C \cdot A}{F_1}$ $357{,}00 \le 10{,}50 + 87{,}68 + \frac{(584{,}57 + 53{,}55 + 70{,}01 - 135{,}60)\tan(30)}{F_1} + \frac{40 \cdot 19{,}94}{F_2}$ $357{,}00 \le 10{,}50 + 87{,}68 + 275{,}45 + 199{,}40$ $357{,}00 \le 573{,}04$	1,2	4	2,2	7,3
B23*	N.A.P. 500 AÑOS (457,46 M) + SUBPRESIONES REALES (MEDIDAS POR PIEZÓMETROS)	$E_{e,l} \le E_{r,l} + P_l + \frac{(P_n + E_{e,n} + E_{r,n} - S)\tan\phi}{F_1} + \frac{C \cdot A}{F_1}$ $357{,}00 \le 10{,}50 + 87{,}68 + \frac{(584{,}57 + 53{,}55 + 70{,}01 - 146{,}67)\tan(30)}{F_1} + \frac{40 \cdot 19{,}94}{F_2}$ $357{,}00 \le 10{,}50 + 87{,}68 + 270{,}13 + 199{,}40$ $357{,}00 \le 567{,}71$	1,2*	4*	2,2	7,3

* Hipótesis con valores de subpresión reales medidas por piezómetros de cuerda vibrante.

Figura 23. Cargas a considerar y coeficientes de seguridad obtenidos.

cargas que contempla la Instrucción, coeficientes de seguridad mayores que los exigidos en ésta (Fig. 23).

9 CONCLUSIONES

Las obras de rehabilitación y mejora de inyección de filtraciones, acondicionamiento del drenaje y control de subpresiones llevadas a cabo en la presa de Portbou han permitido concluir lo siguiente:

– Aunque interese que el nivel de embalse esté por debajo de la cota de una junta fría de hormigonado a tratar, para poder analizar cómodamente el alcance de la resina inyectada, premia el poder trabajar con un nivel de embalse a mayor cota para poder detectar de foma inmediata la efectividad de los trabajos de inyección que se realicen.
– Las resinas epoxi acuaestables empleadas para el tratamiento de juntas horizontales permeables han funcionado satisfactoriamente permitiendo, durante el mismo proceso de inyección de una junta, la gradación a diferentes viscosidades de forma muy simple.
– La inyección a alta presión (hasta 600 bar) de resinas epoxi acuaestables de alta viscosidad es una solución óptima que permite el sellado de las filtraciones y recuperar el monolitismo de la estructura garantizando su estabilidad.
– Con el objetivo de optimizar económica y técnicamente obras de rehabilitación o ejecución de una nueva red de drenaje es imperativo disponer de auscultación piezométrica en la zona a tratar así como investigar el cimiento a cierta profundidad (ensayos geofísicos, cámaras de vídeo, …) para detectar posibles vías preferenciales de circulación de agua.
– A la hora de definir la ubicación de un dren es vital disponer de información sobre la distancia entre la boca de éste y el contacto cimiento-presa. Si esta distancia es mayor que la subpresión medida en esa sección de control, el dren no será efectivo y no logrará disipar subpresiones. Por el contrario, si la distancia es menor, sí será efectivo.
– A nivel de proyecto constructivo de una presa de hormigón (bóveda, gravedad), es importante el diseño de las galerías de inspección de forma que permitan drenar correctamente el cimiento. Hasta cierto grado, el estado de subpresiones puede ser limitado a unos valores máximos aproximados con la construcción, entre otros elementos adicionales, de galerías de drenaje a una distancia de cimiento adecuadas.
– Para el caso de la presa de Portbou, el diseño de la galería de inspección no permite drenar de forma óptima las subpresiones del plano de contacto presa-cimiento aunque sí se limita esa acción inestabilizante a unos valores máximos los cuales no comprometen su seguridad de la infraestructura frente al deslizamiento.

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Corrección de filtraciones en la presa de Horno Tejero (Badajoz)

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ABSTRACT: The Horno Tejero Dam is located on the River Lácara, just up-water from the town of Cordobilla de Lácara, and is therefore classified as a Category A dam, in accordance with Spanish regulations.

It is a rock-fill embankment dam, 37 m high and 300 m wide, with a thick clay core. The shoulders of the dam are made of rock fill, and there is a filter and drain between the core and the rock-fill on the down-water face of the core, 2.5 m wide.

In 2004, seepage was detected in the down-water slope of the dam at a higher than expected level. The consequent analysis of the dam and its construction materials gave rise to concerns that fines were being lost from the core, and that these fines could block the drainage system. This, in turn, would cause the water level in the down-water shoulder to rise.

The solution to the problem was to construct a vertical cement-bentonite slurry wall in the centre of the clay core. In this article, the solution and the construction process are described in detail.

RESUMEN: La presa de Horno Tejero cierra el río Lácara, aguas arriba del casco urbano de Cordobilla de Lácara, en las inmediaciones del mismo; encontrándose clasificada, por tal circunstancia, en categoría A, según Resolución de 29/07/98.

Se trata de una presa de materiales sueltos, de 37 m de altura sobre cimientos y 300 m de longitud de coronación, con núcleo vertical de arcilla vertical, de gran anchura. Los espaldones están formados por un todo-uno, con un dispositivo de filtro y drenaje dispuesto de forma paralela al talud de aguas abajo del núcleo, a continuación de este, con 2,5 m de anchura en su plano superior, dispuesto 4 m por debajo del labio del aliviadero.

En el año 2004, ante la detección de filtraciones en el talud de aguas abajo de la presa, según surgencias observadas a cota superior a la correspondiente al sistema de drenaje dispuesto, una vez analizadas las características de los materiales empleados en la construcción del núcleo y del espaldón de presa, así como la cota superior del indicado sistema de drenaje, se entendió que se estaban produciendo arrastres de finos que, procedentes del núcleo, podían llegar a colmatar el sistema de drenaje, sobrepasando los paquetes formados por materiales drenantes, con riesgo posterior de saturación del espaldón de aguas abajo.

Como solución al fenómeno, se planteó la ejecución de una pantalla vertical de bentonita-cemento, dispuesta en el interior del núcleo.

En el artículo se plantea la solución adoptada para la corrección de las filtraciones, exponiendo el proceso de ejecución seguido, con indicación de los resultados finalmente obtenidos.

1 INTRODUCCIÓN

La Presa de Horno Tejero se sitúa en la cabecera del Río Lácara, aguas arriba de la localidad de Cordobilla de Lácara (Badajoz), en sus inmediaciones; con el dique de cierre en la provincia de Badajoz y parte del vaso de embalse en la de Cáceres.

Construida a finales de los años 80, se trata de una presa de materiales sueltos de 37 m de altura sobre cimientos y 300 m de longitud de coronación, situada a la cota 328 m.s.n.m. Tiene un núcleo grueso de arcilla vertical, y espaldones formados por un todo-uno, con un dispositivo de filtro y drenaje dispuesto de forma paralela al talud de aguas abajo del núcleo, con 2,5 m de anchura total, con su cara superior a la cota 320 m.s.n.m.; el labio del aliviadero se sitúa a la cota 324 m.s.n.m., con el cauce a la 291 m.s.n.m.

Tiene dos galerías, una perimetral y otra longitudinal, alojando esta última los tubos de desagüe de fondo. El volumen de embalse que crea es de 24 Hm^3.

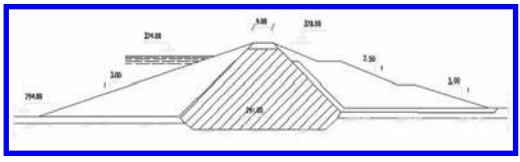
2 PROBLEMÁTICA OBSERVADA

El funcionamiento de la presa ha sido satisfactorio hasta el año 95/96; sin embargo, tras un período prolongado de intensas lluvias, acontecidas a continuación de una serie de años de sequía continuados, se detectaron algunas humedades en las galerías de la presa, así como encharcamientos en el tramo superior del paramento de aguas abajo de la presa.

Las filtraciones aparecidas en las galerías fueron tratadas en el año 2000 por la Confederación Hidrográfica del Guadiana, mientras que la del paramento a la altura de la berma superior se canalizó para su aforo y estudio.

El hecho de que se detectaran las filtraciones en el tramo superior de la presa, en correspondencia con el paso de agua filtrada por el cuerpo de presa a cota superior a la del sistema de drenaje interior de la presa, hacía temer que podrían existir otras filtraciones o zonas especialmente saturadas en puntos a menor cota, donde la existencia de filtro dificulta que estas puedan ser apreciables a simple vista, con riesgo de saturación de este.

Las humedades y encharcamientos observados en el talud de aguas debajo de la presa, tuvieron un seguimiento continuado por parte de técnicos de la indicada Confederación Hidrográfica del Guadiana, disponiendo para su control una red piezométrica, a la vez que requiriendo la colaboración de los órganos centrales del Ministerio de Medio Ambiente. Así, durante los meses de Junio y Julio de 2004 se realizaron dos visitas a la presa por parte de Técnicos de las Áreas de Inspección de Presas y de Geotecnología, teniendo como consecuencia derivada la realización de un estudio geofísico con el fin de determinar el estado de saturación del espaldón de presa y del núcleo.



Así, fue emitido informe por parte de los técnicos personados del Área de Inspección de Presas, en el que se resumió la problemática existente en los siguientes términos:

- La presa está clasificada como CATEGORÍA A (clasificación en función del riesgo potencial del vigente Reglamento Técnico de Seguridad de Presas y Embalses) debido a la presencia de un núcleo de población al pie de la presa, lo que condiciona cualquier actuación obligando a incrementar en lo posible las garantías de seguridad.
- El material con el que se ha ejecutado el núcleo podría presentar una excesiva permeabilidad, mientras que el espaldón de aguas abajo sería, en cambio, excesivamente impermeable.
- Se observan asientos muy importantes medidos en coronación desde la puesta en servicio de la presa, pudiendo haber alteraciones en el interior del núcleo.
- Se estima que el núcleo previsiblemente se encuentra saturado en amplias zonas del mismo.

- Se estima muy probable la existencia de arrastre de finos, a consecuencia de las filtraciones existentes, que podrían estar colmatando el sistema de filtros.

De este modo, vistos los materiales empleados en la construcción del núcleo y ante la perspectiva de que pudieran estar produciéndose arrastres de finos que acabaran colmatando el sistema de drenaje, con el indicado riesgo posterior de saturación del espaldón de aguas abajo, considerando asimismo la proximidad del núcleo urbano

a la obra, se planteó la ejecución de una pantalla vertical de bentonita-cemento, en el interior del núcleo, garantizando de este modo la impermeabilidad del mismo.

3 SOLUCIÓN ADOPTADA

El objeto del proyecto, como se ha indicado con anterioridad, fue la corrección de las filtraciones existentes en la presa, con la consiguiente mejora de las condiciones de seguridad.

Para lograr el objetivo marcado se proyectó, como solución con carácter definitivo, la construcción de una pantalla plástica de cemento-bentonita, vertical, ejecutada desde coronación de la presa, mediante paneles alternos de 800 mm de espesor.

La pantalla se diseñó con el fin de cerrar en su integridad el camino a las filtraciones existentes, previéndose alcanzar el cimiento de la presa, penetrando en el terreno de apoyo, hasta alcanzar roca sana. Así, la profundidad máxima a alcanzar, se estimó en 38 m, con una longitud que ascendió a 283 m.

Como operaciones previas a la ejecución de la pantalla se consideró el corte del tráfico en el vial de coronación, con desvío del tráfico a través del casco urbano de Cordobilla de Lácara, y el desmontaje y demolición de los elementos existentes para la ejecución de los muretes-guía, como es el caso del acerado y de las barandillas de aguas arriba, así como del sistema de iluminación de la coronación de la presa.

Complementariamente, se contempló la ejecución de inyecciones de impermeabilización de la galería de desagüe de fondo, con el fin de impermeabilizar el área no accesible con la pantalla de bentonita-cemento, en la intersección de esta con la galería, cuyo eje es perpendicular al plano de la pantalla., así como las inmediaciones de la galería en la que se alojan los conductos del desagüe de fondo de la presa, dado lo arriesgado de aproximarse con los medios mecánicos de ejecución de la pantalla a dicha galería.

Finalmente, se contempló la reposición del tráfico rodado por la coronación de la presa, lo que se consiguió mediante la aplicación de una capa de aglomerado asfáltico; asimismo, se consideró la integración paisajística y social de la obra, mediante la ampliación del acerado demolido, hasta conseguir un ancho final de tres metros (acerado aguas arriba), facilitando de este modo el paseo vecinal, la creación de una zona de aparcamiento de vehículos en el estribo izquierdo de la presa, la reposición de elementos tales como biondas, alumbrado público, … etc.

4 EJECUCIÓN DE LA PANTALLA DE CEMENTO-BENONITA

4.1 *Operaciones previas*

Las operaciones previas a la ejecución de la pantalla, consistieron en la ejecución de los siguientes trabajos: corte y señalización del desvío del vial de coronación (Carmonita—Cordobilla, (BA-099), cuya titularidad ostenta la Diputación de Badajoz); desmontaje de biondas; demolición de aceras, retirada de barandilla y de bionda, desconexión de

prisma de telecomunicaciones relacionado con el dispositivo del SAIH, corte y demolición de vial para ejecución de muretes guía, relleno del vial de coronación con zahorra para ejecución de plataforma de trabajo y protección del vial, vallado del murete guía como protección frente a caídas a distinto nivel, e instalación de planta de fabricación de la mezcla y adopción de las medidas previstas, relacionadas con las protecciones a tomar en el ámbito de la seguridad y salud laboral.

4.2 *Dosificación de la mezcla cemento bentonita*

Previamente al comienzo de los trabajos se realizaron una serie de mezclas, con distintos tipos de cemento y distintas dosificaciones.

La dosificación empleada en la obra fue definida junto a la Dirección de Obra con base en los ensayos previos de laboratorio realizados. La dosificación finalmente usada, fue la siguiente:

– Agua: 1000 litros
– Bentonita (C-2): 55 Kg
– Cemento (CEM III-B 42,5): 220 Kg
– Aditivos. No se emplearon.

Con la mezcla indicada, se midió una resistencia a compresión simple de entre 6 y 7 Kg/cm^2 y una permeabilidad de 1×10^{-9} m/s.

4.3 *Instalaciones y medios mecánicos empleados*

Las instalaciones de fabricación de la mezcla consistieron en una planta especial capaz de dosificar por peso los tres componentes, agua, bentonita y cemento. Transcurrido el tiempo establecido de mezclado en la planta se mandaba directamente la mezcla a la zona de excavación.

Con este sistema de fabricación, sin maduración de la bentonita, fue preciso utilizar algo más de bentonita que con el sistema tradicional, cuando se fabrica previamente el lodo de bentonita, dejándolo hidratar durante 12 a 24 horas; sin embargo, se consiguió evitar el montaje de una planta de gran volumen, con depósitos de almacenaje de bentonita en maduración, de difícil encaje en el entorno de la presa, sin producir afecciones de todo tipo en el entorno.

En la excavación de la pantalla, se empleó una grúa de celosía LIEBHERR HD-855, y cuchara bivalva de cables STEIN K-610 de 16 t de peso. El empleo de una cuchara pesada, adecuada para excavación en terrenos duros, fue indicado por la necesidad de empotrar lo máximo posible en la roca subyacente a la presa, pero sin emplear el trépano rompedor. El uso del trépano fue desechado por las vibraciones que produce.

4.4 *Ejecución de la pantalla*

Durante la ejecución se controlaron todos los parámetros de la mezcla (ph, densidad, viscosidad y filtrado), con el fin de asegurar su idoneidad. De otra parte, se fabricaron probetas para comprobar la resistencia a compresión simple y permeabilidad conseguidos in situ.

Las características medias del lodo resultante se resumen del siguiente modo:

– Ph: 9–10
– Densidad: 1,17 kg/cm^3

– Viscosidad: 40–44 seg.
– Filtrado: 94–104 cm^3
– Resistencia a compresión simple: 5–7 kg/cm^2 a 28 días
– Permeabilidad: 1×10^{-8} m/seg.

Debido a la naturaleza del cemento-bentonita, las muestras que se tomaron para determinar la resistencia y la permeabilidad en laboratorio, se curaron en condiciones optimas, durante alrededor de una semana, antes de transportarlas a laboratorio, para evitar su alteración.

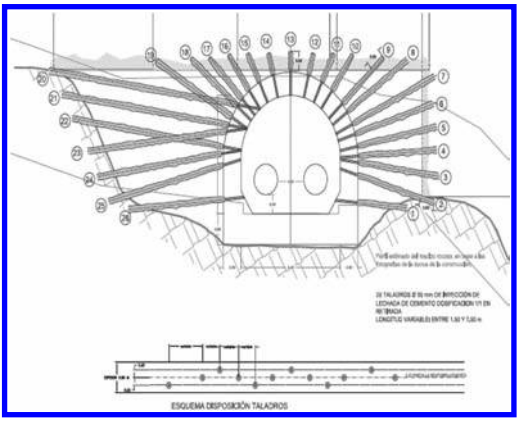
La pantalla se excavó por paneles alternos (primarios y secundarios) solapados entre sí 0,25 m, manteniéndose estables las paredes de la excavación mediante uso del lodo de bentonita-cemento que, una vez fraguado, constituye finalmente la pantalla de impermeabilización.

El rendimiento medio obtenido en la construcción de la pantalla, ascendió a 115 m^2/día.

5 INYECCIONES COMPLEMENTARIAS DE SELLADO

De forma complementaria a la construcción de la pantalla, con el fin de impermeabilizar el área alrededor de la galería, se ejecutó un abanico de inyecciones mediante el empleo de tubos manguitos, con un total de 27 taladros de 56 mm de diámetro, con longitudes variables, comprendidas entre 1,50 y 8,50 m. Finalmente, se completó el abanico anterior con la ejecución de taladros en la solera de la galería para la comprobación de su estado.

El tubo manguito empleado, consistió en un tubo provisto de válvulas anti-retorno de inyección, separadas cada 50 a 100 cm, a través de las cuales se realiza la inyección a presión. Las válvulas anti-retorno se conforman por aberturas en el tubo cubiertas por manguitos de goma que los presionan firmemente y que ceden únicamente bajo presión, y vuelven luego a su posición inicial.



Se inyectó una mezcla con dosificación agua/cemento/bentonita: 1/1/0,035 (1.000 kg de cemento y 35 kg de bentonita por cada 1.000 litros de agua), procediéndose con obturación en boca del taladro a una presión de 5 a 7 kg/cm^2.

Las admisiones obtenidas durante la inyección oscilaron entre 30 y 50 litros por metro de taladro, a una presión de 5 kg/cm^2. En un 25% de los taladros, fue necesario repetir taladros por aparición de goteos debido a la lentitud del fraguado de la mezcla. En estos taladros repetidos, se aumentó la dosificación del cemento y se eliminó la bentonita en la mezcla (lechada a/c:1/2). Además, se incrementó la presión de inyección hasta los 7 kg/cm^2, aumentándose también las admisiones.

6 LOS SONDEOS DE COMPROBACIÓN

Con el fin de conocer el estado de la pantalla plástica en toda su profundidad, una vez fraguada la

mezcla, se realizaron 7 sondeos de comprobación en la misma; lo que permitió reconocer, complementariamente, el estado de la roca inferior a la pantalla.

Los sondeos de comprobación, consistieron en taladros de 100 mm de diámetro, prolongados hasta perforar 1 m en roca sana, con recuperación del testigo, con el fin de comprobar la continuidad de la pantalla plástica.

Asimismo, en cada sondeo, se realizaron dos ensayos de permeabilidad tipo Lefranc; el primero, bajo carga variable, a 2 m del pie de la pantalla y, el segundo, en la roca sana. Los datos obtenidos en los ensayos "in situ" verificaron los ensayos de permeabilidad de la mezcla en laboratorio, del orden de $k = 1 \times 10^{-8}$ m/seg. La permeabilidad de la roca sana se estimó en $k = 1$ a 2×10^{-7} m/s.

En uno de los sondeos, se realizaron ensayos de permeabilidad a diferentes profundidades, para verificar que la permeabilidad de la pantalla fuera uniforme en toda la altura de la pantalla.

7 RESULTADOS Y CONCLUSIONES

En la Presa de Horno Tejero, se ejecutó una pantalla plástica de bentonita-cemento con el fin de eliminar las filtraciones que se habían detectado en el tramo superior de la presa.

La dosificación del lodo empleado fue:

- Agua: 1000 litros
- Bentonita (C-2): 55 Kg
- Cemento (CEM III-B 42,5): 220 Kg.

El resultado final fue una pantalla plástica con resistencia a compresión simple 5 a 7 Kg/cm^2 y permeabilidad del orden de 1×10^{-8} m/seg. Asimismo, se han realizaron inyecciones con tubos manguitos para lograr la impermeabilización del entorno de la galería en la que se alojan los conductos de desagüe de fondo.

Con estas actuaciones, se consiguió eliminar las filtraciones, reduciendo las presiones intersticiales medidas en los piezómetros colocados aguas abajo a la pantalla.

El costo total de las operaciones realizadas, ascendió a la cantidad de 1.977.913,74 €.

Dam monitoring techniques

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Experiences with a 'new means' of leakage detection

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ABSTRACT: This paper describes a technique called Controlled Source Audio Frequency Domain Magnetics to track map and monitor leakage through dams.

The problem most dam engineers face is that leakage can often be traced at the toe of a dam or downstream of it, in some cases upstream within the reservoir basin, but it is usually very difficult to track the leakage through or under the dam.

The 'Willowstick' technique, described in this paper allows individual leakage patterns to be mapped through or under dams.

1 INTRODUCTION

Many dams leak, and thus unless it increases in quantity or starts to take materials with the flow, is not necessarily a problem. However, if some sort of erosion process takes place and the flow increases then failure could occur. Remedial works would normally involve comparatively expensive techniques of grouting or slurry trench valley etc but obviously if the extent and location of leakages can be accurately predicted then the amount of remedial works and thus costs can be limited.

This paper describes the technique and how it was used to locate leakage and the proposed remedial works at a 200 metre high rockfill dam in Sri Lanka.

2 THE WILLOWSTICK TECHNIQUE

Controlled Source-Audio Frequency Domain Magnetics (CS-AFDM), nicknamed Willowstick, is a geophysical technique that uses a low-voltage, low-current audio frequency electrical signal to energise groundwater or seepage flows in the areas of interest. The Willowstick method works by measuring the signature magnetic field response of a controlled, Alternating electric Current (AC) flowing through a specifically targeted subsurface study area—ie the dam and its foundation.

The Willowstick magnetic field is created by a large electric circuit consisting of three parts: (1) the antenna wire connecting two or more electrodes; (2) the electrodes or points of coupling with the earth; and (3) the targeted subsurface study area itself, which is located between and/or around the strategically placed electrodes. The diversity of site conditions in dams often necessitates wide variations of electrode and antenna configurations and interpretive parameters.

For a leaking dam, electrodes are placed upstream and downstream of the embankment or structure. The upstream electrode is placed in the reservoir water at sufficient distance from the dam to allow electric current to spread out in the reservoir before reaching the face of the structure. The downstream electrode is placed in strategic locations (seepages, observation wells, or other downstream locations) to facilitate contact with seepage flowing through the dam. The electrical current follows preferential pathways by concentrating in zones within the saturated subsurface that offer the least resistance through, beneath, and/or around the dam's structure. As the electrical current takes various preferential flow paths through, beneath and/or around the dam, it generates a magnetic field characteristic of the electrical current. This unique magnetic field is identified and surveyed at the ground's surface in a grid pattern using sensitive magnetic sensors.

The horizontal and vertical magnetic field magnitudes are measured at each grid measurement station on the surface of the ground to define the electrical current's subsurface distribution and flow patterns. In nearly all cases, the paths of least resistance for electrical current to follow infer zones of higher porosity within the saturated subsurface. The locations (coordinates) of measurement stations are obtained using a Global Positioning System (GPS) unit and are recorded in a data logger along with the magnetic field data. The measured magnetic data are then processed, contoured, modeled and interpreted in conjunction with existing hydrogeologic information to enhance the characterization of groundwater beneath the area of investigation.

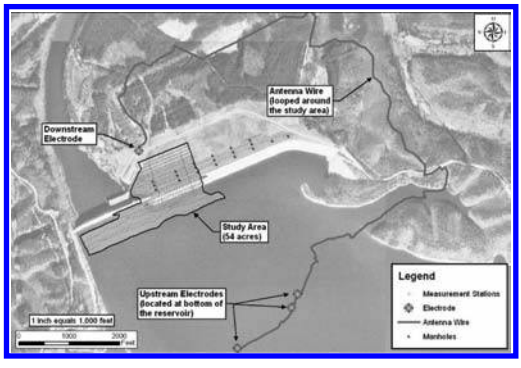

Figure 1. Typical horizontal dipole (plan view).

Figure 2. Willowstick instrument.

The overall approach to the fieldwork involves energizing the groundwater of interest with an AC electrical current with a specific signature frequency (380 or 400 hertz) between the paired electrodes. As the electrical current follows preferential flow paths in the study area between the electrodes, it generates a recognisable magnetic field that is measured by sensitive coils. Magnetic field measurements are generally taken along lines ranging from 5 to 15 metres apart with stations on each line spaced at 5 to 15-metre intervals. These distances vary from one project to another depending upon resolution requirements and other site conditions. The grid pattern proposed for any particular investigation is designed to provide sufficient detail and resolution to adequately delineate the groundwater of interest while at the same time optimizing funds available for the investigation.

Each measurement station's X, Y, and Z coordinates are recorded as part of the field work. The stations are designated by small red crosses or "+" signs shown in the figures. These spatial locations are critical to data processing, comparison of surveys, modeling and interpretation

2.1 *Equipment*

The equipment used to measure the magnetic field induced by electrical current flowing in the groundwater of interest includes: three magnetic sensors oriented in orthogonal directions (X, Y, and Z-axes); a data logger used to collect, filter and process the sensor data; a Global Positioning System (GPS) used to spatially define the field locations; and, a Windows-based handheld computer used to couple the GPS data with the magnetic field data and store it for subsequent reduction and interpretation. All of this equipment is attached to a surveyor's pole and hand carried to each field station (see Figure 2).

The magnetic receiver (nicknamed the "Willowstick") filters and monitors various frequencies, amplifies the signals through noise-reduction algorithms, measures the strength of the signature magnetic field, and converts all the information into a recordable data set that is ready for subsequent processing and corrections (collectively called data reduction).

2.2 *Physical principles involved in Willowstick*

The following principles used by Willowstick show how the technique can be applied to identify, map, and model electric current flow paths that infer groundwater distribution patterns through the subsurface.

- Electric current follows the path of least resistance. Groundwater is generally the best subsurface electrical conductor.
- Electrical current flowing in a conductor generates a magnetic field with characteristics that reflect the location of its source.
- Based on Maxwell's equations, an alternating electrical current in a conductor will generate an alternating magnetic field around the conductor. The converse is also true. An alternating magnetic field will generate an alternating electrical current in a conductor that is under the influence of the alternating magnetic field.
- Two coils in close proximity to each other can be coupled magnetically. A transformer is a special case of two magnetically coupled coils. The electric current in the primary coil creates a magnetic field which then induces an electric current in the secondary coil, completing the magnetic coupling. The primary coil in the Willowstick technology is created by a large primary loop that consists of the antenna wire, electrodes, and the preferential conductive pathways (groundwater) in the subsurface between the electrodes.

The Willowstick technology's secondary coils are in the magnetic receiver. When the magnetic receiver is under the influence of the magnetic field emanating from the primary coil (conductive subsurface flow path), the three receiver coils sense and measure the strength of the magnetic field emanating from the primary coil.

- Conductive features will gather electrical current flowing in the ground. This is referred to as current gathering. Electric current flowing in the ground will follow long conductors or conductive zones that facilitate conduction between point A and point B (ie the two electrodes). The long subsurface conductors or semi-conductors can be formed into four general categories:
  1. The first class of conductors is subsurface groundwater flow paths and channels. When electric current is biased to flow through a subsurface study area, the electrical current will tend to concentrate and flow in the most conductive medium, which is the groundwater/seepage/leakage.
  2. The second class of conductors is 'culture', or any long continuous conductor that is man-made. These include: communication cables, overhead and underground power lines, underground metallic pipelines, metal fences (chain link, barb wire, etc.), railway tracks, steel guardrails, and other elongated continuous conductors. The locations of such are usually known, thus they can be accounted for when interpreting the data.
  3. The third class of conductor is wet clays, which often poste a problem in DC resistivity and other electrical or electronmagnetic (EM) methods. Near surface clays can act as a "shield" and cause much difficulty measuring the electrical properties of materials beneath the clay. The Willowstick methods has two advantages. First, it measures the magnetic field response, which is not directly affected by wet clays as are the electric field or EM field. Second, by directly energizing the medium of interest, greater control can be maintained in most cases to minimise the amount of electric current straying and flowing in the west clays. In most cases, natural subsurface waters moving through a channel will have a lower resistivity than even wet clays, so the current will tend to focus in these if the proper energising perspective is employed.
  4. The fourth class of conductor to consider is any and all other geologic semi-conductors such as graphite-bearing shale, sulphide ores and iron formations. Because of their nature and distribution, these geologic materials are rarely present in significant quantities to cause a problem around Willowstick surveys.

2.3 *Interpretation and modelling*

After data reduction is complete, one or more footprint maps are created to reveal the patterns of electrical current flow in the subsurface by showing contrast between areas of high and low electrical conductance.

When identifying anomalous patterns in a typical footprint map, it is important to be able to distinguish the three main influences on electric current flow. Besides the groundwater influence, the residual effects of culture and electric current bias still exist in most cases—even though the data reduction processes may have reduced these effects to some degree. These three main influences are discussed below:

1. The groundwater influence in a typical footprint map is obviously of greatest importance. As discussed, Willowstick is based on the principle that its signature electric current is strongly influenced by the presence of groundwater, or areas of higher porosity where groundwater is accumulating and/or flowing. The electrical current will naturally gather and concentrate in these areas or pathways of higher conductivity, which are revealed in the footprint map.
2. The magnetic field may be influenced by culture, which is any conductive man-made feature such as pipelines, power lines, or other long continuous conductors. Culture is not always present, but it is often a factor and sometimes very problematic because it tends to be near-surface and can cause large anomalies that mask some of the signal coming from the subsurface. The best approach is to identify all culture before a survey is initiated and either avoid as much of it as possible by strategic survey design or remove its effects when interpreting the data.
3. The magnetic field in any given survey is always subject to electrical current bias because electric current must travel from one electrode to the other in order to complete a circuit. The variable part of the circuit—and the interesting part—is what happens to the electric current when it is allowed to choose its own paths to flow between electrodes. It is always true, however, that 100% of the electric current must concentrate in and out of the points of coupling (the electrodes), and hence the magnetic field tends to grow much stronger as it nears these points.

Interpreting magnetic field contour maps could be compared to reading a topographic quadrangle map. On a topographic map, the ridge lines connecting the peaks could be thought of as the pathways offering the least resistance to traverse. In the same way, these lines in the magnetic field maps

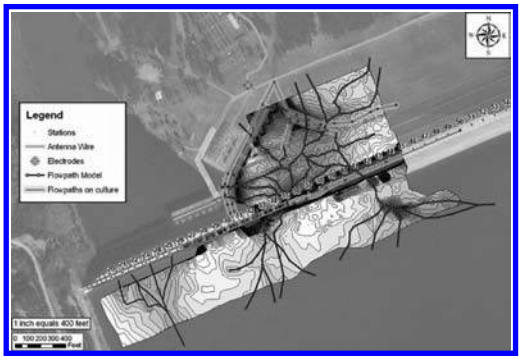

Figure 3. Electric current flow path model.

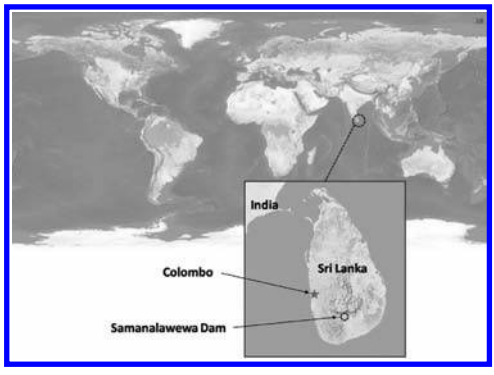

Figure 4. Samanalawewa Dam location map.

represent paths of least resistance for electrical current to follow, although it undergoes some measure of dispersal and regrouping in more complex ways than can be fully described. By identifying these high points and ridges and connecting them together through the study area, the center position of strong preferential electric current flow can be identified (see dark blue lines in Figure 3). Note that the flow paths attributed to culture are highlighted to keep them separate from those attributed to groundwater.

In some cases electric current flow paths produce very tight and revealing anomalies that can be modeled with a fairly high degree of accuracy (depths to within 10% error). Other times electric current flow patterns are not as distinctive and the depth and character can only be roughly estimated—in which case it is very important to have additional data to help characterize the groundwater zone of interest, such as, well logs, piezometric data, or other geophysical or hydrological data. In any case, the horizontal position of electric current flow paths is generally determined with a high degree of consistency and accuracy.

The results obtained from a Willowstick geophysical investigation is to be used to make informative decisions concerning how to further confirm, monitor and possibly remediate groundwater problems through a given area of investigation.

3 EXAMPLE OF THE WILLOWSTICK SYSTEM IN PRACTICE

The Samanalawewa Dam is located in the country of Sri Lanka. Sri Lanka is located off the southern tip of India. The dam is located approximately 160 km southeast of the capital city, Colombo (see Figure 4).

The Samanalawewa Dam impounds the Walawe River, the 5th largest river in Sri Lanka. The

Figure 5. Photograph of Samanalawewa Dam.

Walawe River, in conjunction with another major tributary—the Belihul Oya River—flows from the mountains of central Sri Lanka. The two rivers flow in parallel valleys in a southeastern direction and eventually join together. The horizontal separation of the two rivers is roughly 6 km while the vertical separation between the Walawe River (after joining the Belilhul Oya River) and Katupath Oya (a tributary of the Walawe) is over 300 metres. This difference is used as the head for Power Generation.

The construction of the Samanalawewa Dam was started in 1986 and completed in 1991. The dam and resultant reservoir are one of the largest storage facilities. created in recent times in Sri Lanka. The dam is a zoned rockfill embankment with clay core. The dam is roughly 105 m high and 530 m long and retains a reservoir with a capacity of 254 Mm^3. The catchment area of the dam covers nearly 350 km^2. Not only is the dam important for its renewable energy resource, but it also serves as a key element for water supply, flood control, fish and wildlife and many other immeasurable benefits to the country of Sri Lanka (see Figure 5).

4 GEOLOGICAL SETTING

The project is located within the highlands that lie in the Balangoda region of the Central highlands of Sri Lanka.

The reservoir is situated in the 'Highland Complex' with the underlying rock types comprising metamorphic rocks including granulate gneisses, charnockite, marble and dolomitic marble. These rocks are overlain by a thick weathered layer.

The dam's right abutment and right rim areas consist of karstic terrain. Karst conditions develop from the dissolution of the host rock along fractures, joints and/or bedding planes which become enlarged over time from the saturation and flow of groundwater along these features. In addition to karstic conditions, the right abutment and right rim areas have been subject to extensive folding, faulting and hydrothermal reactions, making the right abutment and right rim areas geologically complex.

During the investigation and construction phases of the reservoir it was recognised that karstic features were likely to be common in the right bank.

The right abutment rises up to a peak of elevation of 545 metres AOD and then descends to a low ridge which extends southwards to form the right bank of the reservoir. The topography beyond the abutment is based on saddles; with four low saddles located on a ridge within a distance of 2.9 kilometres of the dam site. These saddles occur at levels varying from 20 to 60 metres above the top water level of 460 metres AOD.

A karstic feature, a cave, was discovered during the construction phase 300 metres upstream of the proposed axis of the dam on the right abutment. This cave appeared to form along a minor fault, one of a number of minor, parallel faults which create the saddle features noted above.

This and other signs of possible leakage through the right abutment area resulted in an extensive grouting program. Six large cavities, similar to that shown above, were found and sealed with concrete during construction.

5 MEASURES TO LIMIT WATER LOSSES

During the construction of the dam 4 adits were driven along the axis of the dam.

Despite the effort to cut off seepage through the right abutment area, a small spring appeared downstream of the dam upon initial filling of the reservoir (June 1991). The seepage was large enough to suspend filling the reservoir. Additionally, a flat water table was observed responding to the reservoir levels up to a distance of 2.5 km from the dam (along the reservoir's right rim). As a remedial measure, a 1,880 m-long tunnel was drilled beneath the right rim area. From inside the tunnel, a 100 m-deep by 1,600 m-long grout curtain was constructed.

6 'LEAKAGE INCIDENT' & SUBSEQUENT REMEDIATION

On the 22nd October 1992 water burst out of an area downstream of the right abutment of the embankment when the water had reached a level of 439.01 metres OD. The water level was immediately lowered to 430 metres AOD over a period of 3 weeks ending on the 11th November 1992. However, groundwater levels in the right abutment area were kept high as a result of a blockage at the downstream end of the 'karst/pipe feature'. Once this blockage had been removed the ground water level dropped by 2–3 metres in the abutment area. Nearly 25000 m^3 of earth was washed away from the adjacent hillside.

The next remediation effort consisted of installing a dumping of clay from barges in an attempt to slow seepage from flowing out of the reservoir into suspected ingress areas along the noted fault zones. However, after installing nearly 50,000 m^3 of clay, the leak was not stopped. No reduction was noted after the first phase dumping but after the second phase it was reported that the clay blanket did help reduce marginally the groundwater pressure in the right abutment.

7 WILLOWSTICK SURVEY LAYOUT

The Willowstick investigation of the right abutment study area employed one horizontal dipole electrode configuration to energize the subsurface study area. The original scope of work was limited to only one demonstration survey.

Shortly after the fieldwork was initiated and based on preliminary results, Willowstick suggested a second survey. This second survey was targeted to investigate possible seepage through the right rim grout curtain area. This work was limited to a minimal number of measurement stations and was done to investigate whether or not seepage was a problem through the grout curtain. The intention of the additional work was not to detail or model seepage flow paths through the right rim grout curtain. Rather, it was to identify if major seepage path(s) exist through the grout curtain and if so, to determine what additional work should be performed to fully characterise seepage through or beneath the right rim's grout curtain study area.

In performing the two surveys, an injection electrode was placed in the reservoir (some distance from the upstream face of the dam and right rim area). A return electrode was strategically placed in contact with seepage flowing from the hillside down-gradient of the embankment.

Figure 6 presents an interpretation of Survey 1 results. This figure shows the positions of the ECF modelled flow paths (vertical and horizontal alignment).

It appears, based on these posted elevations that seepage north of the tunnel is near the elevation of the reservoir level at the time of the survey (about 440 m). The model also suggests that seepage north of the tunnel occurs above the tunnel and finds an opening in the adit's grout curtain very near the 440 m elevation. Seepage south of the tunnel flows a few metres deeper at elevation 438 m. At the location where the two flow paths converge, east of the adit, the elevation of the preferential flow path drops a little more rapidly as it flows to the discharge point out of the hillside. It is important that all elevations as well as horizontal positions are the result of a relatively inaccurate GPS data set. Nevertheless, the data still provides a good indication of how seepage flows past the dam.

Seepage through the open gap in the tunnel's grout curtain appears to be slightly below reservoir water level as well. Had the grout curtain been placed below the tunnel in the gap, seepage would still have passed through this area because the seepage flow is above the tunnel. Where the grout curtain was placed above the tunnel, just west of the adit's grout curtain, seepage is split by the upward vertical grout curtain and some flows through the adit grout curtain and some flows south of the tunnel around the adit's grout curtain.

In conclusion the results of the investigation suggest that there are a series of braided seepage flow paths north and south of the tunnel that run beneath the right abutment study area. Seepage appears to concentrate around the right side of the dam rather than underneath or through the dam's earthen embankment. There is some seepage occurring along the right rim grout curtain, but not to the extent that it is flowing through the right abutment study area. It has been recommended any seepage through karst topography needs to be carefully characterised, monitored and possibly remediated to insure the integrity of the reservoir as well as the safety of those residing downstream of the dam.

8 REMEDIAL WORKS

The Willowstick survey has confirmed two main areas where the cut-off is compromised—one on the bend of the tunnel and the other where the original cut-off crosses the tunnel. Having identified the location of what we believe to be the two largest sources of leakage through the right abutment area, and it is possible to undertake cost effective remedial works at Samanalawewa Dam to ensure the safety and integrity and to bring the reservoir back to its proposed top water level and retain its ability to generate energy.

The work will involve measures:-

1. To close 2 significant gaps in the existing supplementary grout curtain.
2. To locate and fill major karstic features at approximate elevation of 438 m.

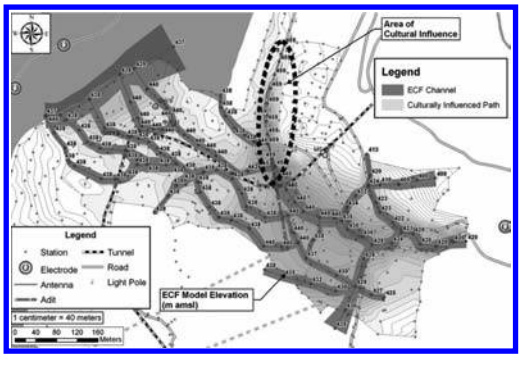

Figure 6. ECF model with posted elevations.

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Levantamiento topográfico mediante escáner láser de la presa de Compuerto (Palencia)

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ABSTRACT: Topographic raising scanner laser of Compuerto's dam (Palencia). Procedure and advantages with regard to the classic topography. Cost and time reduction in the accomplishment of the work. Automatic obtaining of sections to demand, close to orthophotos and map of elevation of colors of the dam. Virtual tour and web publication of the work.

1 INTRODUCCIÓN

En Enero de 2010 se realiza el levantamiento topográfico mediante escáner láser de de la presa de Compuerto en Palencia. Con este levantamiento se pretende mostrar el funcionamiento de esta novedosa técnica rápida, precisa y el registro prácticamente "total" que se genera.

Para muchos levantamientos según ejecución y topografía, los escáneres láser han demostrado que las tareas de campo se reducen considerablemente en comparación con los métodos tradicionales. El escaneado disminuye el tiempo de trabajo de campo mientras que a la vez reduce los costes derivados; la tarea de campo disminuye así a una quinta parte respecto de los métodos tradicionales para muchos proyectos y el nivel de detalle es considerablemente mayor.

2 EL ESCÁNER LÁSER; DEFINICIÓN, TIPOS Y CARECTERÍTICAS TÉCNICAS

Un escáner láser es un aparato topográfico (orientable y referenciable) capaz de registrar puntos en tres dimensiones, con coordenadas xyz mediante un haz láser y el movimiento continuo de rotación horizontal y vertical oscilante de un espejo interno. El instrumento incorpora una cámara fotográfica en su interior que dota de color verdadero a los puntos tomados, de esta forma se analiza el objeto y se reconstruye tridimensionalmente. Estos modelos digitales son utilizados en variedad de aplicaciones desde el análisis de estructuras construidas (as built), pasando por el diseño de prototipos industriales o la documentación de edificios y objetos históricos.

Existen dos tipos de escáneres, los que contactan con el objeto a medir (brazos mecánicos) y los que no necesitan contacto con el objeto. Los primeros se utilizan en la industria para la medición y registro de pequeñas piezas y con ellos se obtiene una gran precisión, incluso de micras, mientas que los segundos son adecuados para medir a grandes distancias y a objetos inaccesibles, por tanto este se tipo de escáner es el que se adapta a nuestro estudio.

Dentro de los escáneres láser, o sin contacto con el objeto, se distinguen dos tipos, los de "tiempo de vuelo", que son capaces de medir la distancia al objeto una vez el haz láser insersecta con el, y los de "luz estructurada", que proyectan un modelo de la luz en el objeto y miran la deformación del modelo en el sujeto a medir. Estos últimos son mas rápidos, si bien generan mas "ruido", entendiendo por ruido puntos que hay que eliminar pues no corresponden al objeto a medir.

En el ejemplo que nos ocupa, se ha utilizado un escáner de tiempo de vuelo; Scan Station 2 de la marca Leica.

También se ha utilizado una estación total Leica TPS 1200 para la toma de bases topográficas y dianas o targets del escáner láser necesarios para la referenciación del escáner y para el posterior postproceso en gabinete del levantamiento topográfico.

Características técnicas generales del Scan Station 2:

1. Campo de visión completo, registra puntos en 360º horizontales y verticales.
2. Compensación de doble eje, como un nivel topográfico, mantiene el aparato totalmente nivelado.

Figura 1. Scan Station 2 de Leica.

3. Precisión topográfica, 4 mm en distancia, aumentado la precisión a 2 mm una vez modelada la presa.
4. Velocidad máxima de registro 50.000 puntos/seg.
5. Alto alcance, registra puntos hasta 300 m de distancia, es el de mayor alcance del mercado, por tanto el mas adaptado al registro de objetos de gran tamaño e inaccesibles como las presas.

3 PROCEDIMIENTO PARA LA TOMA DE DATOS Y TRATAMIENTO DEL REGISTRO OBTENIDO

1. Se confecciona en los alrededores de la presa una red de bases topográficas constituida por clavos de acero colocados en las carreteras de acceso a la presa aguas arriba y aguas abajo. Estas bases nos permitirán relacionar los distintos escaneados necesarios para registrar toda la presa y referenciarlos tanto a los vértices geodésicos como a las bases con las que se ejecutó la construcción de la presa, de tal forma que se pueda comparar sus desviaciones. A su vez se colocarán las dianas reflectantes utilizadas por el escáner. Dichas dianas serán tomadas por la estación total TPS 1200 de Leica junto a las bases topográficas situadas en torno a la presa. El escáner se sirve de las dianas o targets para obtener la orientación del levantamiento.
2. Por medio del escáner, "Scan Station" de Leica, se realizan varios escaneados desde distintos puntos o posicionamientos para poder registrar totalmente la presa, al ser imposible un único registro, por las forma, dimensiones de la presa y por las dificultades orográficas del terreno que la circunda.

Figura 2. Escaneado aguas arriba.

En este ejemplo se realizan 3 posicionamientos:

1. Escaneado aguas arriba.
   Nota: Este escaneado registra la presa desde la lámina de agua hacia arriba. Para registrar mayor zona de la presa seria necesario que el nivel de agua descendiera, pues el haz láser no puede atravesar el agua y es reflejado quedando sin registrar la zona sumergida.
   El agua en movimiento por el contrario si puede ser registrada, como sucede en el aliviadero, ya que el haz no es reflejado por las gotas de agua.
2. Escaneado aguas abajo.
   Con este único escaneado se toma la mayor parte de la construcción aguas abajo quedando muy pocas zonas de "sombra" o sin escanear.
3. Escaneado sobre el nivel de la presa, en su margen izquierda.
   Desde esta posición se domina la práctica totalidad de la superficie de la coronación de la presa.
4. Unión de escaneados gracias a un sofware específico. En primer lugar el programa toma como referencia las bases registradas en campo y las dianas reflectantes del escáner láser. Posteriormente se unifican los diferentes escaneados, con sus respectivas dianas y con las coordenadas de la dianas.
   Se genera un modelo tridimensional proveniente de los tres posicionamientos con una densidad 2 cm máximo.
   De forma automática se enlazan las nubes de puntos de los diferentes posicionamientos, generando una única nube de puntos (Figura 5). Una vez obtenido el modelo tridimensional, se genera una malla (superficie triangulada). A partir de esta malla se pueden obtener secciones longitudinales, transversales y en planta, a la distancia que se desee. En este ejemplo se realizaron secciones cada dos metros.

Figura 3. Escaneado aguas abajo.

Figura 4. Escaneado sobre el nivel de la presa.

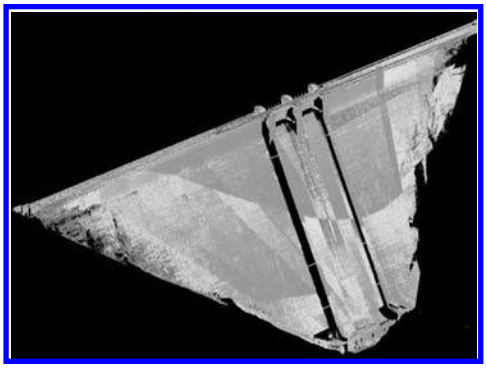

Figura 5. Nube de puntos antes de aplicarles el color verdadero.

4 RESULTADOS OBTENIDOS

Aparte de los planos tradicionales obtenidos del modelo digital, en planta, alzados y secciones con gran nivel de detalle, se generan otro tipo de representaciones que se detallan a continuación y que complementan y mejoran el nivel de comprensión de la presa.

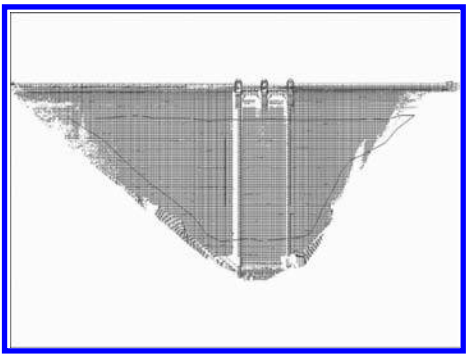

Figura 6. Cuadrícula de las secciones cada 2 metros horizontales y verticales.

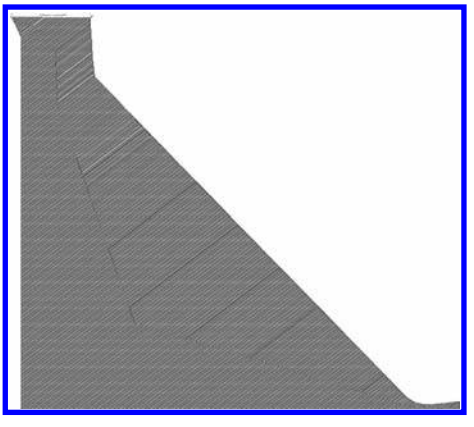

Figura 7. Ejemplo de sección vertical.

4.1 *Ortofotos*

Como ya se indicado anteriormente, el escáner genera una nube de puntos con color proveniente de la su cámara interna, por ello se pueden generar ortofotos con color verdadero de la presa en las que se pueden localizar patologías tales como grietas, desconchones, cambios de color, vegetación, etc … Las deficiencias del paramento se pueden localizar con coordenadas precisas o superficiar. Dichas ortofotos se realizaron a partir de la malla generada de la nube de puntos tridimensional y fotografías externas tomadas de la presa para mejorar la calidad de los fotoplanos. Se dotó de color a la nube de puntos y se generó posteriormente la malla con color.

En la Figura 8 se puede observar la ortofoto de una parte de la presa.

4.2 *Mapa de desviaciones de colores*

A continuación se comparó el plano principal de la presa aguas abajo con un plano teórico ajustado por el software del escáner para ver las

Figura 8. Ortofoto.

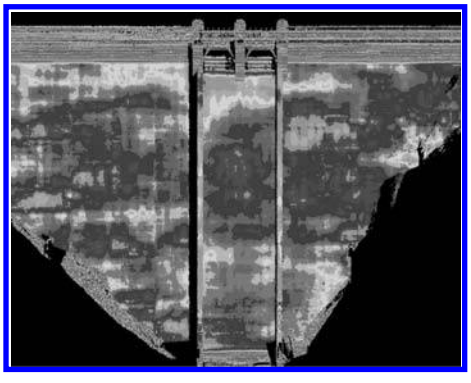

Figura 9. Mapa de desviación de colores.

desviaciones respecto a la forma teórica, generando un mapa de desviaciones. Dicho plano se generó con una escala de colores, siendo cada color el intervalo de dos centímetros. De este modo se pueden ver y localizar rápidamente los posibles desperfectos de la presa.

4.3 *Reproducción del modelo, mediante recorridos virtuales y visualizadores interactivos*

Finalmente se produjo un video de un vuelo virtual sobre la presa, recorriendo las diferentes estructuras de la construcción. Dicho vuelo se realizó a partir de la nube de puntos obtenida del escaneado láser.

También se creó en formato HTML un visualizador interactivo desde el cual se puede consultar medidas situándonos en los diferentes posicionamientos realizados en campo con el láser escáner.

La visualización tridimensional es una forma de publicitar o difundir los resultados obtenidos del levantamiento de la presa, ya que hace esta sea más comprensible para la mayoría del público.

5 VENTAJAS DE LA TÉCNICA DE ESCANEADO LÁSER EN LA MEDICIÓN DE PRESAS

1. *Registro rápido y masivo de puntos*
   El escáner láser realiza un barrido masivo de puntos, millones de puntos en pocas horas, con coordenadas espaciales xyz. Estos pueden estar tan próximos entre sí como se desee, teniendo en cuenta que la precisión en distancia es de 4 mm.

   El aparato incorpora una cámara fotográfica que dota de color y textura al modelo digital generado a base de los puntos tridimensionales muy próximos, con lo que se pueden obtener alzados con ortofotos, en ellos se detectarían además de la forma, dimensiones de la presa y sus posibles deformaciones, otras patologías como humedades, grietas y fisuras.

   Del registro de puntos se obtiene un modelo digital tridimensional métrico, a partir del cual se generan plantas, alzados, secciones longitudinales y transversales y tantos planos como sean necesarios.
2. *Registro basado en la medición láser*
   No se necesita contacto con el objeto a medir, por lo que se pueden registrar áreas inaccesibles que de otro modo nos sería imposible llegar a ellas, tales como zonas con fuertes pendientes o taludes verticales.
3. *Registro topográfico*
   El escáner láser como instrumento topográfico que es, se puede referenciar, es decir que el modelo tridimensional obtenido se puede "fijar" sobre el terreno por medio de referencias permanentes (clavos, estacas, etc.) lo que nos permitirá detectar posibles desplazamientos o deformaciones de la presa en un periodo de tiempo determinado, así como la variación en la forma de las patologías.
4. *Reducción de tiempo*
   El trabajo realizado en campo, disminuye con respecto a un levantamiento convencional, lo que repercute en costes y tiempo de tratamiento de los datos obtenidos.

6 COMPARACIÓN CON OTRAS TECNICAS DE REGISTRO TOPOGRAFICO

1. *Topografía clásica*
   Con el escaneado láser se genera un registro muchísimo mayor de la construcción o del terreno, démonos cuenta que el registro normal de puntos realizado con escáner genera una densidad (cuadricula) de puntos de 1 a 2 cm, con la topografía clásica este registro seria inviable económicamente y en tiempo.

Con la topografía clásica las secciones hay que tomarlas insitu, y previamente prefijadas, en muchos casos no es tan evidente saber lo que se esta tomando visualizándolo desde grandes distancias.

Las curvas de nivel que se obtienen en años sucesivos corresponden exactamente a la forma real y no es una interpolación de un programa de curvado, por lo que se ven cambios reales superiores a 4 mm, superponiendo ambos planos.

Con la topografía clásica no se pueden generar ortofotos ni modelos virtuales.

2. *Fotogrametría*

La fotogrametría aporta, al igual que el escaneado láser, planos mas completos que con la topografía clásica y también aporta ortofotos, si bien el escaneado láser tienen las siguientes ventajas.

1. Mucha mayor rapidez en la obtención de datos, y muchísima mayor rapidez en el procesado de los mismos. El operador en fotogrametría debe ir restituyendo punto a punto cada sección, en el caso del escáner, esta realización es automática.
2. La medición en el escaneado es automática y objetiva, es decir el aparato hace la medición con una precisión determinada, mientras que en el caso de la fotogrametría, la restitución es subjetiva, depende de la experiencia del operador, de su capacidad de visión estereoscópica, y su habilidad para el posicionamiento en tres dimensiones, por tanto el error cometido depende en gran medida del factor humano.
3. La obtención de ortofotos, en fotogrametría solo es posible de zonas planas de la presa, no de la totalidad de la construcción, ya que los elementos que no se hallen en el mismo plano quedan distorsionados al transformar la perspectiva cónica de la foto en ortogonal, mientras que en el caso del escaneado, los puntos con color se adaptan perfectamente al modelo tridimensional (la fotografía de adapta al modelo como si fuera una piel), y con el punto de vista ortogonal que se seleccione, se obtiene la ortofoto de todo el objeto.

7 CONCLUSIONES

El escáner láser se está utilizando en la toma de datos de edificios. Se Pueden comprobar las diferentas respecto a su definición geométrica teórica, y seguir su evolución.

Por ejemplo, en monumentos artísticos se pueden medir los desplomes, asientos y otras deformaciones, y hacer un seguimiento periódico de los mismos.

En edificios industriales se pueden tomar datos de todos los elementos situados en su interior, como es la maquinaria, tuberías, conductos de ventilación, etc, en un plazo muy breve, interfiriendo un tiempo mínimo en su actividad.

El escaneado de la Presa de Compuerto ha permitido disponer en un plazo muy breve de un levantamiento topográfico de las misma muy preciso, con todas sus irregularidades, como son erosiones, pérdidas de material por efectos meteorológicos, humedades, etc, con una precisión topográfica una vez modelada la presa de 2 mm. De este escaneado se pueden sacar los planos necesarios en cualquier formato convencional.

Repitiendo el escaneo posteriormente, con un coste bastante inferior ya que parte del trabajo hecho para el primer escaneo es válido, se puede comprobar la evolución de las patologías del hormigón, midiéndolas con precisión. Se podrá comprobar en cuánto se han prolongado las fisuras, las pérdidas de material, etc, pero lo más importante, medirlas.

Es posible también escanear el interior de las cámaras, e incluso de las galerías. En las cámaras aparecerían representados todos los elementos allí instalados con la misma precisión, como son válvulas, by-pass, puentes grúas, aducciones de aire, cuadros eléctricos, etc, y también las irregularidades, como fisuras o humedades.

Utilizando los datos obtenidos, es rápido y sencillo generar una imagen 3D de la presa, e incluso de su interior, pudiendo movernos por ella con libertad. Hay que tener en cuenta que la imagen que se genera procede además de los puntos escaneados que la sitúa, de imágenes fotográficas, por lo que la representación será real.

En la reparación de paramentos en presas, será de gran utilidad hacer un escaneo previo. Una vez reparado el paramento, cualquier anomalía que se produzca en el nuevo paramento podrá situarse con mucha precisión sobre el paramento antiguo.

En las figuras 10 y 11 se puede ver el paramento de aguas arriba de la presa de Arlanzón. Aunque afortunadamente se ha comportado bien, cualquier anomalía posterior se podría relacionar y superponer con el paramento antiguo.

En las figuras 12 y 13 se pueden ver los paramentos de la misma presa aguas abajo.

En la Confederación Hidrográfica del Duero se va a utilizar este sistema para hacer un levantamiento topográfico de un talud inestable, en la margen izquierda del río Carrión en Carrión de los Condes, que puede afectar a una ermita del siglo XVI situada próxima al borde superior del talud. El acceso es difícil y peligroso, ya que se puede producir un deslizamiento en cualquier momento. Se pretende tener información del mismo

Figura 10. Presa de Arlanzón. Paramento antiguo, después de sanearlo.

Figura 11. Presa de Arlanzón. Paramento nuevo.

Figura 12. Presa de Arlanzón. Paramento de aguas abajo antes de la reparación.

Figura 13. Presa de Arlanzón. Paramento de aguas abajo después de la reparación.

con mucha precisión en el estado actual y una vez estabilizado, sin necesidad de acceder al mismo. Posteriormente se podrá hacer un seguimiento periódico de su comportamiento.

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Laser scanning and digital imagery as a tool for dam monitoring and rehabilitation

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ABSTRACT: Research, miniaturization and lowering costs have made available both terrestrial laser scanners and high resolution digital cameras. The fusion of data recorded by these Combined Terrestrial Imaging Systems (CTIS) can provide the dam engineer with an unprecedented amount of information. After processing the combined data, accurate three-dimensional models, including pictorial information with metric quality, can be generated. These models can be used in different phases of the dam lifespan, from the excavation of the foundations, to the comparison between as-designed and as-built objects as well as to the monitoring phase. Regarding dam surveillance CTIS can be used both for deformation monitoring and for assisted visual inspections in a unique field campaign. As for geometric monitoring, Laboratório Nacional de Engenharia Civil (LNEC) has developed a methodology, accessories and software to quantify both the 3D displacement of materialized surface marks and the deformation of the walls of the dam, or its surroundings. The uncertainty and reliability of this method are comparable to the traditional tacheometric methods which measures only points that have been materialized with targets. Concerning the information traditionally acquired during visual inspections, LNEC has developed methodologies that allow the codification in office of the deterioration symptoms into a database. The information gathered in this way has proven to be more accurate, comprehensive and objectively classified than the information collected using the traditional approach. As far as rehabilitation is concerned, this technology can produce as-is 3D CAD and texturized models which will provide valuable information for refurbishment acts that might enlarge the life of the dam or for recovering lost functions after an event. Both raw data and deliveries are electronically recorded, processed, stored and managed, facilitating their migration to IT systems where the information can be quickly retrieved or the evolution of the deteriorations can be tracked. The same information can be exchanged over the internet among dam owner, consultants, authorities, supervisors, designers and contractors. Costs and delivery time are comparatively lesser than using the traditional methods, taking into account the quality and quantity of the information gathered. Besides reporting on the application to dam surveillance and rehabilitation, of this already proven technology, this article will highlight, based on experience, some details that should be further researched to improve the developed methodologies.

1 INTRODUCTION

Terrestrial laser scanners create a low cost and accurate 3D geometric model of an object. This very dense model is composed of millions of points whose coordinates are gathered at a frequency up to one half million points per second. If a digital calibrated camera is attached and referenced to the laser scanning system it is possible to enhance the geometric model with radiometric information.

The fusion of these two types of information, coming respectively from the Terrestrial Laser Scanner (TLS) and from the camera, gives rise to a 3D photograph with metric quality. This photo-realistic model of the object can be explored by the dam engineer in a manifold manner.

On the scope of dam engineering the information delivered after processing the raw data provided by these Combined Terrestrial Imaging Systems (CTIS) can be used in several phases of the life span of a dam.

The unprecedented amount of geometric and radiometric information can be used just as a record, e.g. of the foundations, should one need to revisit them after the dam was built, or to map and track the evolution of the visible deteriorations of an ageing dam. Reverse engineering activities can be supplied with as-is CAD models,

for instance having in mind refurbishment or maintenance.

This paper will report applied research and uses of this multi-sensor technology, regarding dam surveillance and rehabilitation. After this introductory section a second one outlines briefly the technology regarding the laser scanning system and the photo sensing camera. A third section of this paper will describe the processing phases, illustrate some 2D examples of outputs and enumerate the 3D outputs which can only be fully explored in a computer monitor. Both types of outputs will concern different applications. Having in mind the huge amount of information which is collected in a very fast field operation, some specifications should be clearly stated prior to the data acquisition phase and are outlined in section four. Finally, and before the conclusions, some research and development needs will be emphasized, based on experience.

2 BACKGROUND

Regarding the theoretical background of the technology and as far as fundamental formulae are concerned the reader is referred to the bibliography, namely the books in the references section of this paper.

Because CTIS are multi-sensor platforms both sensors are briefly and separately approached in the following subsections.

2.1 *Laser scanning*

Laser (Light Amplification by Stimulated Emission of Radiation) mechanisms emit electromagnetic radiation, usually visible monochromatic light which is spatially coherent and has a low divergence beam.

When in pulsed operation, high peak powers can be achieved and the time that takes the pulse to travel from the emitting component to the sensing component of the same TLS can be converted into a distance (ρ) to the reflecting object point P (Figure 1).

The pulsed beam can be mechanically and incremen tally deflected of a specified angle both in a, say vertical, plane and in an orthogonal plane, say horizontal. This means that for every pulse that was emitted and reflected, one gets, in addition to the intensity of the reflected pulse (I), the 3D polar coordinates of the reflecting point. These polar coordinates can be transformed into a Cartesian coordinate system associated to the instrument. After transformation one gets a cloud of points whose 3D coordinates have been computed along with the respective intensity values, i.e. (x, y, z, I). Because the TLS is an active sensor

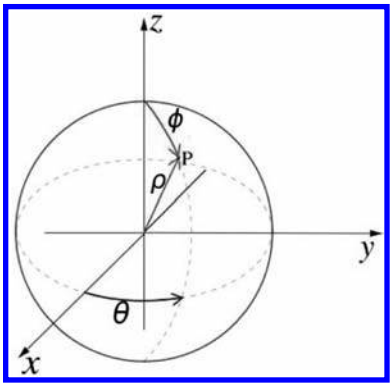

Figure 1. Terrestrial laser scanners measure the magnitude (ρ) of a vector deflected of θ and ϕ which are converted into Cartesian coordinates with an instrumental origin.

I can be recorded at night and one may get an intensity map which very much resembles a gray-scale photograph. Either using indirect referencing, via known control points, or direct referencing, if the scanner has some orientation and positioning capabilities, a new transformation matrix can be computed in order to convert the instrumental coordinates (x, y, z) into an object related—and more meaningful—set of coordinates (X, Y, Z). (Lerma, 2008).

2.2 *Photo sensing*

Photo sensing elements arranged in an array can sample the object through a lens system, in a spatially and radiometrically discretized manner. The resulting 2D digital image (i, j, R, G, B) is a file that contains the Red, Green and Blue intensity values of a scenario discretized in pixels which are located in line i and column j of the sensor array. The digital camera is calibrated to take into account the effects (δi, δj) on the image of the geometric distortions of the lens system and to compute the transformation parameters that relate (i, j) with the camera related photo-coordinates (u, v) (Berberan, 2003).

2.3 *Data fusion*

From the 2D undistorted digital image, which is a function of the type

$$G(i,j) = \left(g_r(i,j), g_g(i,j), g_b(i,j)\right) \quad (1)$$

where g_r, g_g and g_b are discrete functions with the intensities of the three, red, green and blue, visible wavelengths of the electromagnetic spectrum, and using the colinearity equations.

$$u = c\frac{r_{11}\Delta X + r_{12}\Delta Y + r_{13}\Delta Z}{r_{31}\Delta X + r_{32}\Delta Y + r_{33}\Delta Z}$$
$$v = c\frac{r_{21}\Delta X + r_{22}\Delta Y + r_{23}\Delta Z}{r_{31}\Delta X + r_{32}\Delta Y + r_{33}\Delta Z} \quad (2)$$

where u and v are the photo-coordinates, c is the focal distance, r are the nine elements of a three dimensional rotation matrix relating photo and object coordinates and ΔX, ΔY and ΔZ are the three components of a known vector connecting the origin of the photo-coordinates to point P it is possible to get a texturized point cloud

$$G(X,Y,Z) = \left(g_r(X,Y,Z), g_g(X,Y,Z), g_b(X,Y,Z)\right) \quad (3)$$

which is basically a file with a (X, Y, Z, R, G, B, I) type of organization.

3 FROM PROCESSING PHASES TO DELIVERIES

The very high frequency of data acquisition gives rise to a huge amount of data in a short period. This means that specification, planning, processing and quality control phases should be highly regarded. Technical specifications are dealt with in section 4. The processing phases will be dealt in this section along with the presentation of several practical examples and different ways to systematize the information in engineering documents.

3.1 *Processing phases*

The raw data is collected in separated point clouds depending on object size, surrounding topography, scanner range, obstructions, etc. A pre-processing phase produced in site the texturized point cloud.

Several phases will take place, before this data is converted into engineering documents. The degree of automation of these phases is considerably lower than the data acquisition phase, meaning that the number of days needed to process data will be greater than the days spent on the field. How greater, depends on the type and quality of the delivered documents.

3.1.1 *Data cleaning*

A considerable amount of noise exists among the millions of points recorded due to many factors like temporary or permanent obstructions, multi reflections, bad surface reflection, etc. Data cleaning is achieved using a manual approach, when the operator can identify points that are not related to the object under study (scaffolding or machinery, for instance) or automatic filtering algorithms like the median, to get read of outliers. In both cases care must be taken when removing noise from the data.

3.1.2 *Resampling*

The resolution of the collected points varies due to the angular type of deflection of the laser beam and this might lead to oversample the scenario foreground and to undersample its background. Considering the huge amount of data to be processed it is advisable to get rid of points that are not adding information to the model, as the processing speed, data transfer and data storage will be affected. Care should be taken in order to not loose detail on the representation of the surveyed object.

3.2 *Practical examples and engineering documents*

Laboratório Nacional de Engenharia Civil (LNEC) has carried out some applied research on the use of combined terrestrial imaging systems and advised the owners of dams to use the methodology when appropriated. In these cases terms of reference have been produced for several phases of the life of different types of dams.

The results can be presented in several types of engineering documents. Traditional 2D engineering documents, like cross-sections, profiles, contours, etc., actually representing 3D objects, can be generated automatically from the models. Other new types of engineering documents, which are 3D virtual representations of the objects, can only be fully and interactively explored in a computer platform due to their tridimensional nature. Some examples of the deliverable engineering documents, with different levels of processing, are shown here.

3.2.1 *Geometric and radiometric record*

The recording, with geometric and referenced radiometric information, of a situation that will not be seen ever more is an invaluable asset, should one need to revisit the foundations. Figure 2 shows an unprocessed point cloud that can be seen and measured in site, in nearly real time.

3.2.2 *Reverse engineering and refurbishment*

After the processing and post processing phases several deliveries can be produced in a more or less automatic procedure, yet always on line. Figure 3 shows a 3D model of the downstream wall which is partially texturized, and partially represented by 3D CAD primitives as well as the contour lines of the downstream river banks. A TIN (Triangular Irregular Network) is shown on the right side of the dam wall. This network of triangles is the actual "skeleton" of the 3D model, an intermediate

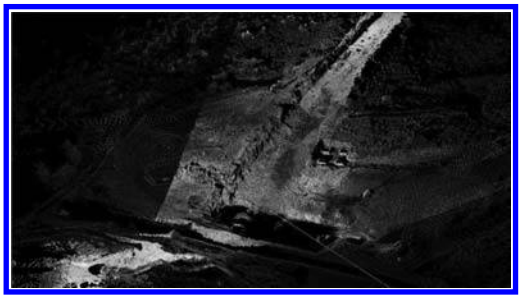

Figure 2. Point cloud with 1 to 2 cm accurate 3D coordinates of the foundations of an embankment dam with a resolution of 1 point every cubic decimeter.

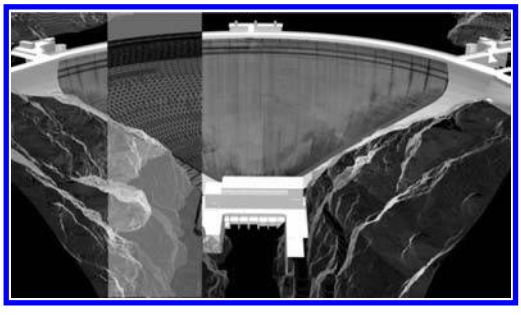

Figure 3. Different presentations of the same 3D model of Cabril dam (Courtesy of EDP—Electricidade de Portugal).

product in the processing of the data towards the final deliveries.

3.2.3 *Visual inspection of the dam wall*

Visual inspections are expensive and time consuming yet very important. The traditional approach to visual inspections is human intensive and has to be performed in site by skilled staff. The result is very subjective, not comprehensive and has no positional accuracy due to operational difficulties.

Figure 4 shows an orthoimage which is an image with a scale, can be loaded to a CAD file and used to map the visible deteriorations on the dam, without operational and personal safety problems.

Under the guidance of one senior engineer several less skilled staff can survey simultaneously several dams, in different computers and this capacity can be of major importance following earthquakes or other extreme natural conditions that might affect simultaneously several dams. The mapping of deteriorations (Figure 5) follows a catalog of deterioration symptoms that has to be defined in advance. The result is validated in site and with the assistance of local staff.

Figure 4. Orthoimage of the downstream wall of Cahora Bassa dam (Courtesy of HCB).

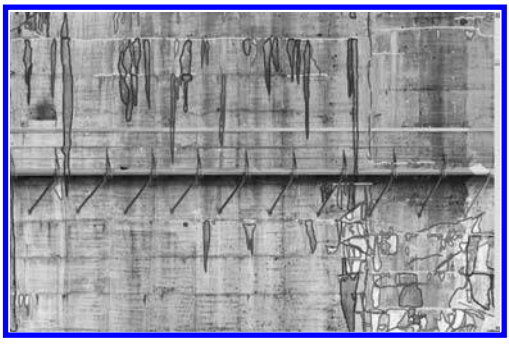

Figure 5. Mapping of deteriorations symptoms using an orthoimage loaded into a commercial CAD software.

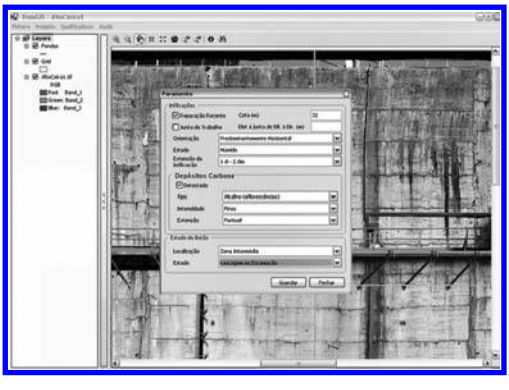

Figure 6. Symptoms represented graphically by points, lines or polygons are associated with their alpha-numeric attributes.

The mapped symptoms can be associated with their meaningful attributes (Figure 6) and populate an information system. In this IT different types of observations coming from several monitoring

instruments are queried and cross checked to evaluate the dam condition or behavior.

The procedure to map and classify deteriorations will vary significantly depending mostly on the resolution of the images as well as on the available software.

3.2.4 *Deformation monitoring*

Laser scanning has a linear uncertainty which is similar to the uncertainty associated to the total stations used for geodetic monitoring. Angular accuracy is improving to comparable standards. In practical terms this means that deformation vectors with a magnitude of a couple of centimeters are detectable, provided that the appropriated equipment and methodology were chosen.

After an abnormal behavior in Lapão dam, three scannings were carried out in 2003, 2005 and 2007 and compared to the design. The results of a particular cross-section are shown in Figure 7. The settlements have been measured also on surface marks using both laser scanning and geodetic observations. The root mean square error was found to be less than 1 cm, in each of the three Cartesian components of the displacement vector.

Epoch-Suite (www.epoch-suite.com) set of programs has been used to manage and adjust the geodetic observations. This suite of programs administers a data base with observations and results in a epoch-by-epoch approach, uses robustified techniques to detect blunders among the observations and graphically displays the history of the displacement of surface marks. The uncertainties of the displacements are presented both in a numerical and graphical manner.

4 TECHNICAL SPECIFICATIONS

The amount of data collected in a short time requires detailed specification, from the consumer side, and a careful planning, from the producer side. The aim of specifying is to allow the comparison between distinct epochs, establish appropriated quality parameters, define acceptance/rejection criteria and homogenize data supports and formats in order to migrate data among users, platforms, networks, applications and software.

The items to specify should include metadata, area and object of surveying, restrictions to operation, reference system and its materialization, targeting, restrictions to electronic data processing methods, chronologic resolution (if for monitoring purposes), spatial resolution and radiometric resolutions, minimum range and maximum beam divergence as well as other technical specifications of the equipment, calibration and referencing procedures, quality control and deliveries. Because Laser can be harmful to health the Class of Laser allowed should be stated. The service provider should supply the user with appropriated software viewers as 3D engineering documents can only be seen in computers.

Shape can be detected in the point cloud and detail can be identified on the image depending on the point density and image resolution, respectively.

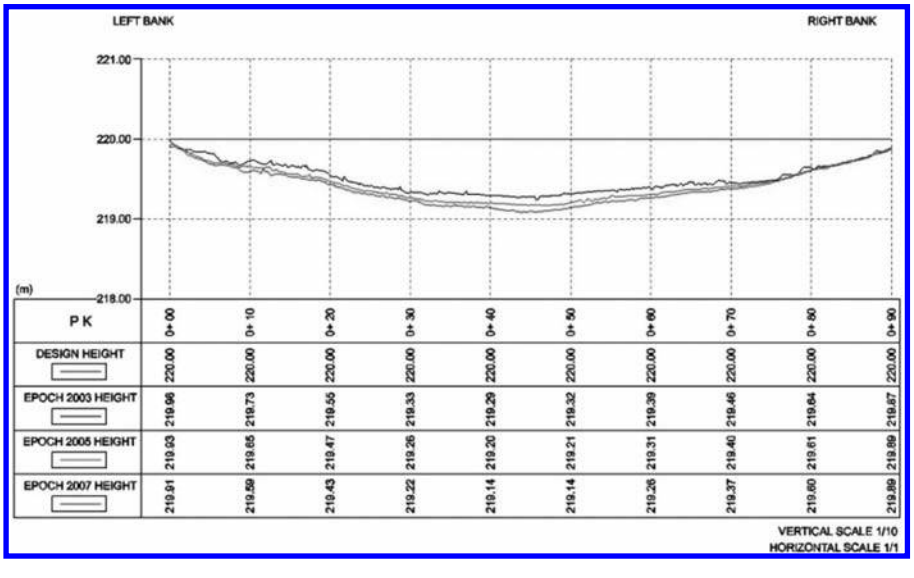

Figure 7. Settlements observed in a particular cross-section: 3 epochs and the original situation.

Table 1. Laser sampling density and image resolution.

Laser	Probability of detecting shape	
Shape dimension (λ)	0.66	0.95
	Point density	
	λ/3	λ/10
Digital imagery	Probability of detecting detail	
Detail dimension (δ)	0.66	0.95
	Resolution (on scenario)	
	$\delta \times 3$	δ

Table 1 gives values that might be used to achieve the desired results.

5 RESEARCH AND DEVELOPMENT NEEDS

Combined terrestrial systems have been commercially available for the last two decades and are a reliable and proven technology used in a myriad of applications. But it is also a technology that is under development meaning, first, that there is still continuous improvements in terms of accuracy, resolution and frequency of data collection, second, that there is also a need for algorithm and software development, and third, that there is still no consensus on the best design or best technology to be incorporated for this equipment. This third item means that it is difficult to specify error models and comprehensive calibration procedures. In these fields research has been carried out (Staiger, 2005) and needs to go on. The use of infrared sensors should be investigated as a tool to help on the classification or detect deteriorations.

On the visual inspection application of this technology, research on automatic or semi-automatic extraction of deteriorations is being done (e.g. Chen, 2006; Yamaguchi, 2008) and results are expected to improve soon.

A last note goes to a variety of 3D electronic engineering documents which can be seen only on electronic displays and that can be used to the advantage of the engineer, in addition to the traditional 2D documents. Once the texturized models have been generated, it is possible to make them available in 3D CAD files, 3D PDF files and VRML files. Defining a path, it is possible to produce also video reports.

6 CONCLUSIONS

To set up a unit to acquire and process 3D models needs large initial investments in laser scanner, camera and lenses, high processing speed computers with large data storage capacities and good graphic cards, software copies and maintenance licenses, as well as training which is significantly costly. On the other hand a cost benefit analysis is clearly favorable in terms of quantity and quality of the gathered information if the workload keeps the unit busy. If this is not the case, the current status of the development of the technology and associated initial investment, points to outsourcing this type of activity.

Combined terrestrial imaging systems have shown to be able to assist visual inspections with different levels of support. Either mapping the deteriorations on top of an orthoimage, using commercial CAD software, or with software designed to extract those deteriorations on a semi-automatic way, this process should always end up with a in situ validation.

The quality of data is expected to improve as laser scanners, which were not developed in a geodetic environment, are now incorporating know how that is common among manufacturers of total stations. The positional uncertainty and range of laser scanners is now sufficient for most engineering tasks that need a high frequency positioning system. These tasks include deformation monitoring provided the magnitude of the displacement vectors is larger than half centimeter.

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Estudio de la dinámica de la calidad del agua, presa Carreta en Tepatitlán de Morelos, Jal. México

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RESUMEN: Los ríos, arroyos, embalses, lagos o lagunas. En el momento en que van a ser destinadas a abastecimiento de aguas potables, deben mantener unos parámetros al menos mínimos de calidad que aseguren su correcto estado. En este siglo 21 la humanidad encara la creciente demanda de alimentos, por lo tanto los ingresos per cápita (Freeman *et al*., 2005).

La presa "Carreta" es fuente de agua potable para habitantes de Tepatitlán, Jalisco, México. Requiere conocerse la variación de los parámetros fisicoquímicos y microbiológicos del agua.

En esta investigación se realiza, monitoreo con muestras de agua en 5 puntos de control del embalse y análisis en el laboratorio del Centro Universitario, mediante el equipo Hydrolab se realiza control *in situ* a diferentes profundidades. Valores de temperatura, conductividad eléctrica, pH, cloruros, NH_4, NO_3, OD, dureza, ST, SVT y óxido-reducción. Algunos marcan datos fuera de rango, según la norma. Agua de regular calidad y resultados microbiológicos indican una grave contaminación, se considera relevante dar a conocer los resultados, para conjuntamente buscar estrategias de acción.

ABSTRACT: All rivers, creeks, lakes, and any body of water needs to be examined for a minimum level of water quality before they're considered as a possible resource for potable water. In this century 21 human kind faces a growing demand of food supplies, therefore the per capita income (Freeman *et al.*, 2005).

The "Carreta" Dam currently provides drinking water to the communities of Tepatitlan, Jalisco, Mexico. It is necessary to know the variation of the physiochemical and microbiologic parameters of its waters. This research monitors several water samples from 5 control points throughout the dam at different depths, then it describes several lab results of these samples which were done using Hydrolab equipment on site. Among the variables are temperature, electric conductivity, pH, chlorides, NH_4, NO_3, OD, hardness, ST, SVT and rust reduction. Some of these variables are out of the limit for an acceptable range for drinking water. Therefore, it is concluded that this water has high microbiologic pollution levels, it is relevant to share this results with other agencies in order to reach to an action plan.

1 INTRODUCCIÓN

En la actualidad, algo menos de las dos terceras partes del agua destinada al consumo humano procede de aguas continentales superficiales, o sea, ríos, arroyos, embalses, lagos o lagunas. El resto se divide entre un tercio de aguas subterráneas y una pequeña cantidad de agua de mar. En el caso de las aguas continentales superficiales, en el momento en que van a ser destinadas a abastecimiento de aguas potables, deben mantener unos parámetros mínimos de calidad que aseguren su correcto estado. Con el objetivo de controlar que ningún vertido industrial y/o urbano al cauce pueda alterar las condiciones del agua, ésta deberá ser periódicamente analizada.

En este siglo 21 la humanidad encara la creciente demanda de alimentos, pero manteniendo la sustentabilidad de los recursos naturales, pues en los próximos 40 ó 50 años se espera se duplique la demanda de alimentos, el crecimiento de la población, el urbanismo y el incremento de los ingresos per cápita (Freeman *et al*., 2005). Tal desarrollo debe fundamentarse en el manejo adecuado a los recursos naturales por dos factores ya evidentes: los efectos negativos del uso de la tierra más allá de su potencial sustentable y la escasez del recurso tierra per cápita (Richters, 1995). Cuando ocurre

sobreexplotación de la tierra, la degradación de los recursos naturales se puede percibir en dos formas: en el sitio y fuera del sitio (Dregne, 1995).

El daño en el sitio impacta al productor agropecuario reduciendo el rendimiento del cultivo o ganado, o incrementando el costo por el uso extra de insumos como fertilizantes, plaguicidas, labranza, irrigación, entre otros. El efecto fuera del sitio se percibe con la sedimentación de canales de drenaje, embalses de presas, eutrofización de cuerpos de agua superficial, contaminación de aguas subterráneas, entre otras. Este impacto se refleja en la calidad del agua a escala nacional, donde el 58.44% de las estaciones que miden la contaminación en las doce regiones hidrológicas de México presentan la condición más común como "agua contaminada" (SEMARNAP-CNA, 2006). A escala local también se ha identificado este problema, como en la cuenca el Jihuite, en Tepatitlán, Jalisco (Flores *et al.*, 2007). El origen de tal contaminación son los sistemas de producción que operan en las áreas de drenaje, aportando grandes cantidades de nutrimentos y agroquímicos, que tienen como destino los cuerpos de agua superficial. Este tipo de contaminación se denomina como Contaminación No Puntual.

La contaminación por fuentes no puntuales resulta de actividades realizadas en la explotación de tierras, tales como la producción de cultivos y generalmente, aguas que penetran en la superficie del terreno como el escurrimiento. Las descargas no puntuales son intermitentes y ocurren debido a eventos meteorológicos (Loehr, 1984), por lo que no pueden ser identificadas a un solo origen de descarga, lo que crea los problemas tanto de identificación como de evaluación del impacto potencial de la fuente no puntual de contaminación (Deliman y Leigh, 1990).

Sharpley et al. (1999) mencionaron que desde finales de 1960, las fuentes puntuales de contaminación de agua han sido reducidas debido a la relativa facilidad de identificación y control. Aún así, los problemas de calidad de agua se mantienen, pero ahora la atención está dirigida hacia a contribución de fuentes no puntuales por el daño que ocasionan a la calidad de agua, problema que se puede visualizar de manera precisa por la muerte de 6% de peces, ocurrido en Estados Unidos. De este valor el 4.4% se atribuye a los insecticidas, 0.1% a fertilizantes y 1.5% a los desechos de estiércoles (Loehr, 1984). En muchas partes de México, se desconoce la situación que guardan los desechos agropecuarios con la calidad del agua.

Williams et al. (1995) mencionaron que los estudios de campo para agroquímicos en el escurrimiento han seguido tradicionalmente uno de tres diseños basados en la extensión del área de drenaje. Uno de los diseños involucra parcelas de prueba de poca extensión del orden de decenas de metros cuadrados. Este diseño se ha utilizado extensivamente para evaluar el escurrimiento y erosión potencial de suelos específicos y pendiente del terreno. La atención en este diseño ha aumentado, para evaluar el escurrimiento con nutrimentos y plaguicidas, y definir las Mejores Prácticas de Manejo para calidad del agua y control de la erosión. El segundo diseño involucra áreas de drenaje más grandes de 1 a 10 ha; es común utilizarla enlazada con estanques de granjas, cuando se requiere someter un plaguicida a registro. El tercer diseño se utiliza cuando se requiere monitorear la calidad de agua ambiental, enfatizando la toma de muestras al azar de grandes reservorios de agua, del orden de cientos de kilómetros cuadrados.

Las razones anteriormente expuestas manifiestan la necesidad de monitorear a través del tiempo la calidad del agua del embalse de la presa La Carreta, pues la experiencia en otras embalses ha mostrado una gran dinámica en la calidad del agua (Ramírez *et al.*, 1996), la cual por ser una fuente de agua potable para habitantes de Tepatitlán, Jalisco, requiere conocerse como es la variación de los parámetros fisicoquímicos y microbiológicos del agua, ya que observaciones eventuales de este vital líquido en la citada presa, se perciben malos olores, presencia de sedimentos y presencia de algas, que permite deducir una hipótesis de calidad de agua, no apta para consumo humano.

2 OBJETIVOS

GENERAL: Evaluar la dinámica de las características físicoquímicas de la calidad del agua de la presa La Carreta, en el municipio de Tepatitlán de Morelos Jalisco.

- PARTICULAR: Revisión y aplicación de métodos de análisis fisicoquímicos para calidad de agua según la NORMA OFICIAL MEXICANA NOM-127-SSA1-1994, "SALUD AMBIENTAL, AGUA PARA USO Y CONSUMO HUMANO-LIMITES PERMISIBLES DE CALIDAD Y TRATAMIENTOS A QUE DEBE SOMETERSE EL AGUA PARA SU POTABILIZACION".
- Monitoreo de la calidad del agua en la presa "La Carreta".
- *Análisis in situ* del agua de la presa "La Carreta".

3 MATERIALES Y METODO

El área de estudio

El proyecto realiza en la presa del predio denominado "el durazno" o "Carreta" ubicado en el km 9.5 carretera Tepatitlán—Yahualica. Jalisco, México.

Metodo y frecuencia de muestreo
El muestreo se realiza mensualmente. Con muestras por duplicado de cada punto. Se baso en dos estrategias; uno, en la parte superficial del acceso al dren de los canales del escurrimiento y otro muestreo, en el perfil en la cortina de la presa *in situ* a una profundidad de cada metro desde la superficie al fondo. Con el equipo Hydrolab DS5X, los parámetros son: Temperatura, pH, NH_4, NO_3, Cl^-, Oxigeno Disuelto (LDO), potencial de oxidación reducción, concentración total de gases disueltos.

Toma de la muestra de agua para el laboratorio
Se tomaron muestra en cinco puntos en el embalse de la presa. Uno en la cortina, en la mitad del embalse y en el extremo superior, dren de agave, de granja y de la línea eléctrica para análisis microbiológicos y fisicoquímicos; sólidos totales, volátiles y dureza.

Metodos y frecuencia de muestreo utilizado
Se toman las muestras de agua de la presa y se llevan de inmediato al laboratorio, colocando las muestras en digestión ó se congelan por un tiempo no mayor de 48 horas. Realizando los análisis en el laboratorio de la Universidad de Guadalajara—Centro Universitario de los Altos de Jalisco.

4 RESULTADOS Y CONCLUSIONES

Con respecto al objetivo específico sobre la calibración de métodos de análisis fisicoquímicos para calidad de agua según la NORMA OFICIAL MEXICANA NOM-127-SSA1-1994, "SALUD AMBIENTAL, AGUA PARA USO Y CONSUMO HUMANO-LIMITES PERMISIBLES DE CALIDAD Y TRATAMIENTOS A QUE DEBE SOMETERSE EL AGUA PARA SU POTABILIZACION".

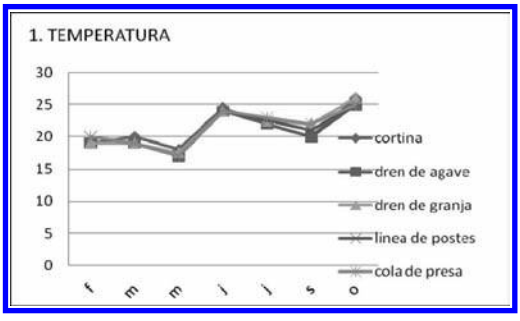
Se implemento en el laboratorio de análisis fisicoquímicos las pruebas analíticas de: temperatura, pH, sólidos totales, sólidos volátiles totales y dureza, se compro materiales para la determinación del DBO5 y se está probando y revisando la técnica.

INTERPRETACIÓN

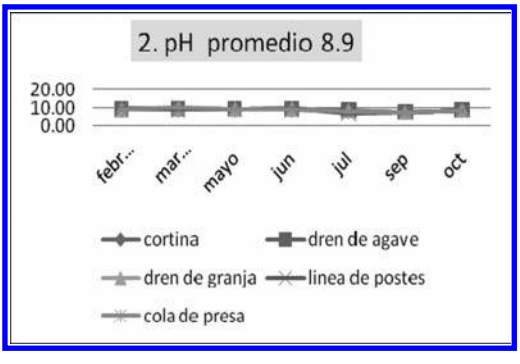
La temperatura; y juega un papel importante ambiental para la producción de vida en el agua, y el comportamiento de este parámetro nos expone temperaturas regulares ambientales constantes y características de la zona (Gráfico 1.). Promedio fue de 21.3°C. Aunque se debe considerar que el aumento de temperatura disminuye la solubilidad de gases (oxígeno) y aumenta en general la de las sales. Aumenta la velocidad de las reacciones del metabolismo, acelerando la putrefacción. Se observa en los resultados un ligero aumento de temperatura, esto inclusive considerando la baja de temperatura ambiente.

Las aguas naturales pueden tener pH ácidos por el CO_2 disuelto desde la atmósfera o proveniente de los seres vivos; por ácido sulfúrico procedente de algunos minerales, por ácidos húmicos disueltos del suelo. La principal substancia básica en el agua natural es el carbonato cálcico que puede reaccionar con el CO_2 formando un sistema tampón carbonato/bicarbonato.

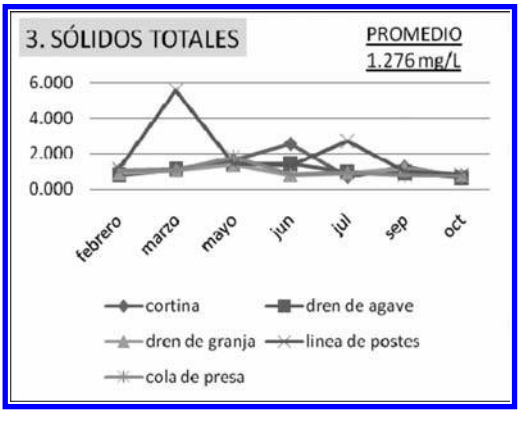
Las aguas contaminadas con vertidos minerales o industriales pueden tener pH muy ácido.



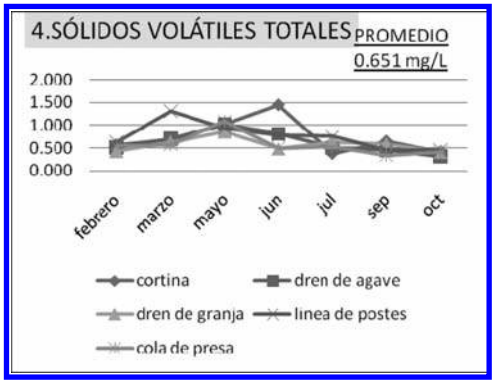
El pH (Gráfico 2.) es de gran influencia en los procesos químicos que tienen lugar en el agua. La NOM 127 nos menciona de un pH de 6.5 a 8.5 aceptable. En la presa se tuvo un promedio de 8.9, observando en los resultados un pH alcalino comúnmente en el agua de la presa, lo que nos indica posible contaminación principalmente de minerales.



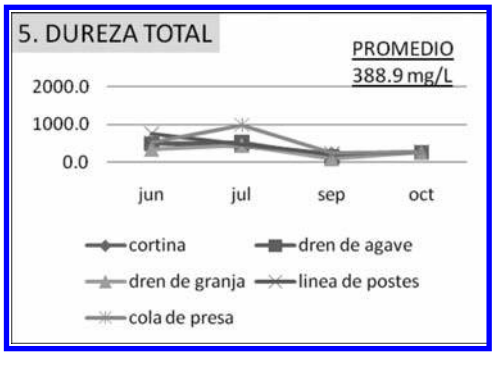
Todos los contaminantes del agua, con excepción de los gases disueltos, contribuyen a la "carga de sólidos". Pueden ser de naturaleza orgánica y/o inorgánica. Provienen de las diferentes descargas, domésticas, de campo e industriales. Encontramos en nuestros resultados especialmente en marzo una gran descarga de sólidos orgánicos.



El contenido de *sólidos volátiles* (Gráfico 3.) se interpreta en términos de materia orgánica. Ésta se oxida formando el gas carbónico y agua que se volatilizan. Sin embargo, la interpretación no es exacta puesto que la pérdida de peso incluye también pérdidas debido a descomposición o volatilización de ciertas sales minerales como por ejemplo las sales de amonio o carbonato de magnesio. Considerando nuestros resultados, se observa valores altos y variables, por lo tanto genera conocimientos de las altas descargas de sólidos orgánicos promedio de 0.651 mg/L.



El análisis de *dureza* (Gráfico 4.) en el agua, mide la presencia de cationes Ca^{+2} y Mg^{+2}, y en menor cantidad Fe^{+2} y Mn^{+2} y otros alcalinotérreos. El contenido de estas sales, determina la "dureza" del agua. Los resultados nos reportan datos variables, con los cuales observamos la constante de contaminantes al agua. Aunque estas sales disueltas en el agua no constituyen un factor de peligro para la salud humana, presentan serios inconvenientes en los procesos de limpieza, ya que se combinan con el jabón formando sales insolubles que impiden la formación de espuma y lo que es más importante, disminuyen la disponibilidad del jabón, aspecto fundamental en los procesos de lavado. Son perjudiciales también, en instalaciones industriales que requieren el uso de calderas, ya que al calentar o hervir este tipo de agua se producen costras de carbonato de calcio $CaCO_3$.



Con respecto a los objetivos específicos de:

- Monitoreo de la calidad del agua en la presa "Las Carretas".
- Análisis *in situ* del agua de la presa "Las Carretas".

Obtuvimos resultados que establecemos comentarios a continuación:

El cambio de *temperatura* disminuye tanto al acercarse el invierno, como en su profundidad de la presa y relaciona todos los parámetros con respecto a la dinámica del medio ambiente.

El *pH* nos indico, cambio de alcalino a acido, variable por las características de la conductividad eléctrica, ionización del agua y el potencial oxido reducción, esto lo genera la biomasa de los sedimentos que se acumulan especialmente en el tiempo de aguas.

Sólidos totales. Los resultados en el embalse, con valores altos, denotan el asentamiento de sedimentos acarreados, generando descomposición observada en los datos NO_3, NH_4, OD y ORP.

Conductividad eléctrica. Indica un nivel constante de la ionización por sales, estos datos tienen relación directa con la dureza y sus interferencias al realizar el análisis.

OD. Nos da valores relacionados directamente con la temperatura, el OD es mayor con menor temperatura, sin embargo nos genera datos de OD bajos con respecto a la profundidad, creciendo los de NO_3 y NH_4.

Los resultados analíticos en general, nos denotan valores altos de pH sólidos y dureza con respecto a los parámetros normados y considerando que la alcalinidad, pH y dureza del agua pueden afectar la calidad del agua para el consumo humano y el uso agrícola, a sabiendas que cuando se encuentra en ciertas concentraciones y combinaciones, la alcalinidad, pH y dureza, pueden incrementar las incidencias de enfermedades cardiacas y otras dolencias (Ortega- Guerrero *et al*., 2002).

Los resultados *in situ*, nos presupone consideraciones de contaminación, esto mencionado desde una perspectiva analítica fisicoquímica, debido seguramente a las descargas sin control. Mismas que son de impacto en las explotaciones lecheras afectando a la calidad de la leche, mediante el uso del agua en la presa, vista como alimento para el ganado, y en el proceso de limpieza de los hatos.

Nos faltaría considerar la parte media del embalse, la cual nos generaría datos relevantes comparativos, El trabajo es arduo, pero consideremos que es la punta de flecha para dar a conocer resultados y la necesidad de concluir este interesante trabajo; el cual nos generará la iniciativa de buscar a más corto plazo estrategias de acción conjuntas, la universidad, empresarios, productores y gobierno.

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Advanced techniques for dam monitoring

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ABSTRACT: Measuring and monitoring deformations of man-made objects and infrastructures is a key task of applied geodesy and geomatics. This paper describes two novel remote sensing techniques based on radar sensors which might make a contribution to dam monitoring. The first technique uses data captured by a Synthetic Aperture Radar (SAR) sensor to derive 2D imagery. This imaging system can be used to provide displacement estimations with sub-millimetre precision through interferometric techniques. The second technique, which employs Real Aperture Radar (RAR) data captured by a similar device but with a different configuration, can be used to measure and monitor the vibrations of man-made objects and infrastructures as a 1D profile. Two experimental campaigns carried out with a ground based SAR and a RAR to monitor a Dam and two man-made structures respectively are described in this paper. Both techniques demonstrated to be powerful and efficient tools.

1 INTRODUCTION

The rapid development of space technology in the last twenty years has allowed the detection of the displacement of Earth surfaces from space with high precision, with unexpected benefits for Earth observation and related global studies. This progress has been possible thanks to microwave images obtained through Synthetic Aperture Radars (SAR) mounted on satellites and the development of advanced interferometric processing tools (Ferretti et al., 2007). These techniques have been later applied to develop ground based instruments aimed at the observation of larger scale phenomena as instable slopes (Tarchi et al., 2003), single glaciers and other applications. The first experiment based on the use of a Ground Based SAR interferometer (GB-InSAR) was devoted to monitoring a dam (Tarchi et al., 1999). Instead of exploiting the forward motion of the spacecraft to synthesize the image, a GB-SAR makes use of the motion of a coherent radar along a mechanical rail, usually 2–3 m long. Maps of the displacement component of an observed surface along the sensor–target Line Of Sight (LOS) can be obtained using the interferometric information contained in SAR images acquired by a portable GB-SAR installed in a stable area, provided that propagation does not introduce decorrelation, namely a loss of information, in an irreversible way.

The rapid growth of microwave technology in the last years encouraged the development and manufacture of different instruments (Bernardini et al., 2007, Pipia et al., 2007, Rudolf et al., 1999). The above-mentioned instruments are developed from advanced close-range remote-sensing radar techniques. Regarding the conventional contact sensors, such as strain-gauges, the use of non-contact radar-based techniques overcomes some limitations and drawbacks. A GB-SAR interferometer supplies a continuous displacement map of the entire observed area up to hundreds of thousands of square meters. Microwave interferometers can be used both day and night and they are negligibly affected by weather conditions. Furthermore, the possibility of operating even at a significant distance and without the need of installing and wiring sensors permits the investigation during emergency situations, when the monitoring activity can be required, to ensure the safety of people.

In this paper the capability of SAR and RAR techniques are discussed on the basis of some experimental data acquisitions, to provide a comprehensive assessment of their pros and cons for man-made structures monitoring. Specifically, the monitoring of a dam during a two-day observation period and the detection of the wind induced oscillations of a tower and a building during a few minutes lapse of time are briefly reported.

2 THE PROPOSED TECHNIQUES

2.1 *SAR Interferometry*

The energy transmitted by conventional radar is concentrated into a beam with an angular dimension, the field of view, ruled by the antenna and basically determined by the ratio between the operating wavelength and its mechanical size. In a radar image, targets that differ from each other in their

azimuth coordinates only, generate overlapping radar echoes and thus they cannot be distinguished (i.e. they belong to the same radar-bin, the elementary volume seen by radar). Conceptually, azimuth location can be achieved by changing the viewing angle of a very directive antenna with high costs in terms of mechanical performances of the system and poor performances. In order to produce a good azimuth resolution, short ranges and large antennas are needed. The SAR technique finds a solution to this problem taking profit from the fact that viewing a target during the entire time it is within a beamwidth, makes it possible to synthetize an artificially long antenna. If the amplitude and phase of the echoes are acquired (through coherent radar) and specific "focusing" algorithms are used, an artificially narrow beamwidth can be achieved. Radar imaging from satellite uses the forward motion of the spacecraft to simulate a much longer antenna, which in turn, provides a higher ground resolution.

It is also possible to acquire SAR images through a portable SAR. The motion for synthesizing the SAR image is obtained linearly, moving the microwave transceiver. The best performance of ground-based radar installations is usually achieved when small scale phenomena (i.e. buildings, small urban area or single hillsides) are monitored. On the other hand, satellite radar is able to monitor a very large area as a drainage basin. The coherent radar images acquired at different dates can be processed to retrieve the interferometric information, φ, when the decorrelation among different images is maintained low.

Interferometry is a technique that uses the phase information obtained from the interaction of two different waves to retrieve temporal or spatial information on the wave's propagation. In the simplest case, when sources of phase noise, namely atmosphere influence and variations on observed surface scattering characteristics, are negligible, the LOS displacement occurred at a pixel between two or more consecutive images, Δr, can be recovered from the measured interferometric phase, φ, according to the following equation:

$$\Delta r = \frac{\varphi}{4\pi}\lambda \tag{1}$$

where λ is the wavelength corresponding to the operating frequency of the radar.

Images acquired from space are also affected by the topography of the Earth's surface due to the different positions of the satellite along the orbital motion (Ferretti et al., 2007). The ordinary configuration of a GB-InSAR provides a "topography-free" interferogram whose phase can be directly related to terrain movements. It is possible to estimate the variation in LOS distance between the radar and the observed surface in different periods of time with a precision in the order of fractions of the operating wavelength which, in the case discussed in this paper, means a sub-millimetre precision.

Interferometric measurements can be also affected by phase wrapping, i.e. the phase ambiguity induced by the modulo 2π phase registration (Ferretti et al., 2007). The ambiguity has to be removed during the processing by adding the correct integer number of 2π to each measured value. Correction techniques for this problem are available but, considering the short time of acquisition of a GB-SAR, the observed motion in many cases can be adequately sampled by the systems and this problem is of minor concern. In the case of experimental campaigns when the system is mounted and dismounted, by repositioning the GB-SAR, this is not able to detect, unambiguously, displacements corresponding to more than π in terms of phase. A solution to this problem could be to integrate the GB-InSAR system and other ranging sensors featuring lower precision and higher measurement rate.

In ground based observations, compared to satellite sensors, it is necessary to find a site with good visibility and from where the component of the displacement along the LOS can be properly observed. On the other hand, satellite observations are sometimes not fully satisfactory because of a lengthy repeat pass time or a problem arising from the observational geometry. Satellite, airborne and ground based radar interferometry are derived from the same physical principles but they often exhibit specific problems mainly due to differences in the geometry of the observation.

2.2 *RAR*

The same principle governing microwaves interferometry applies to situations when the radar acquisition can be very fast. In these cases, a variation of displacement occurred in very short lapse (milliseconds) can be observed using a RAR. In other words, the system is able to measure vibration of the observed surface. To obtain fast measurements the sensor motion must be discarded, loosing the SAR imaging capability. The acquisition of a single time profile is shortened down to a few milliseconds making possible to record the temporal behaviour of structures as bridges, buildings, towers, etc. The temporal behaviour of single radar bins can be analyzed obtaining a deep understanding of the observed phenomena.

3 THE INSTRUMENT

A commercial GB-SAR interferometer is available at the Institut de Geomatics: the IBIS-L constructed

and marketed by IDS (Ingegneria dei Sistemi SpA). The system is designed and manufactured providing the possibility to use the radar sensor both as a RAR device, mounted on a tripod for dynamic measurements (IBIS-S), and as a linear SAR interferometer, moving along a metal rail with 2D imaging capabilities.

The IBIS-L microwave interferometer is a Ku-band radar sensor designed for simultaneous monitoring of the response of several points belonging to a large structure, providing for each point the displacement between two different times. The radar system simultaneously measures the displacements of all points in the area illuminated by the antenna beam with precision up to 0.1 mm. IBIS-L provides also detailed information on the movement of specific points belonging to the scenario, because the position along range is stored in a database for each control point. The sampling rate might be up to an image per 3 minutes. This capability allows the continuous deformation monitoring of structures with fast displacements. In addition, the IBIS-L system can be accurately repositioned in different periods of time, in case a periodic control in a long-period is needed.

The main operational characteristics of IBIS-L system are summarized in Table 1.

Table 1. IBIS-L main characteristics.

Parameter	*Value*
Operating frequency	17 GHz (Ku band)
Maximum operational distance	4 000 m
Displacement sensitivity	0.1 mm
Spatial resolution @ 1 Km	0.5×4.5 m
Acquisition rate	5 minutes
Weight of the whole system	130 kg
Power consumption	100 W

Figure 1. A picture of the IBIS-L system including: the radar module; the linear scanner (2.5 metre long); the control PC and the power supply unit.

The sensor set-up of IBIS-S can be varied to provide the dynamical use with the necessary performances. In particular, the cross range spatial resolution, without synthetic aperture in cross range becomes about 20º. At the same time, by not using any mechanical movement, the measurements are strongly improved from a temporal sampling point of view: a maximum acquisition rate of 100 Hz makes it possible to record the temporal behaviour of structures as bridges, buildings, towers, etc. A photo of the IBIS-L system is shown in Fig. 1.

4 EXAMPLE OF APPLICACTIONS

4.1 *GB-SAR*

An example of an experimental campaign for dam monitoring using GB-SAR is briefly reported here. For an exhaustive description we address the readers to the original paper by Alba et al. (2008). An arch-gravity dam was investigated by a team of the Politecnico di Milano, DIIAR, Polo Regionale di Lecco, Italy and by IDS Ingegneria dei Sistemi using a IBIS-L system to measure deformations due to the filling of the water basin or other factors as thermal variations. The dam, managed by the A2A company, was monitored during a 37 hours time lapse starting on May 28, 2007. The Cancano dam is located in Alta Valtellina, Italy and features about 90 m height from bottom to top, and 350 m length on the crest. A precise Digital Surface Model (DSM) of the structure, generated by TLS and a continuous control by topographic techniques, allowed performing a fine GB-InSAR data validation.

The data was acquired using IBIS-L at a mean distance of 400 m from the dam mid-point, with the radar system placed just in front of the dam's downstream face. This positioning allowed capturing the entire dam displacement along the range direction during the whole observation period. The scanning of the whole downstream takes less than 10 minutes, and each scanning cycle was repeated without intermediate breaks. The main results of GB-InSAR measurements are 2-D deformation maps of displacements which reached a maximum value of about 4 mm in the central section. By determining a threshold both on the basis of their coherence and their Signal-to-Noise Ratio, it is possible to select those points which are highly reliable and which can act as accurate control points for validation. If the selection of the thresholds is restrictive, the precision in the measurement of the displacement of the selected point is higher. In Fig. 2, a sub-set of points with high coherence are depicted on a power radar image; in particular,

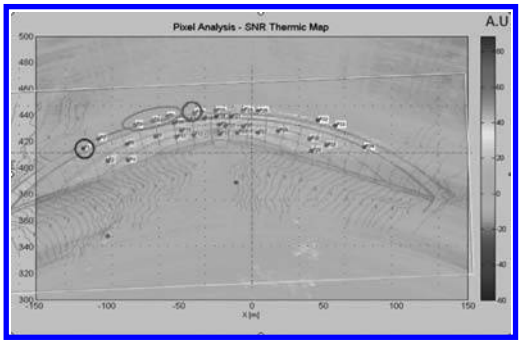

Figure 2. The identification of the high quality points on a power SAR image acquired in front of the Cancano dam. (after Alba et al., 2008).

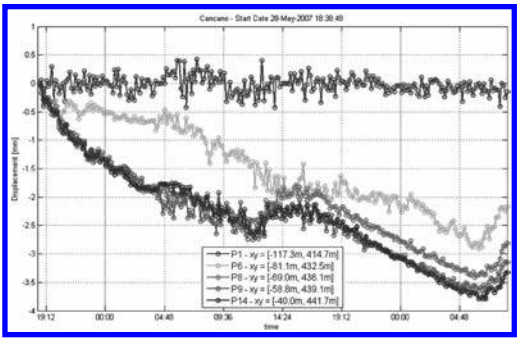

Figure 3. Temporal profile of the displacement measured at five points located on the dam crown.

the position of five points is enhanced with circles of different colour: blue for P1, orange for P6 and purple for P8, P9 and P14.

Five points belonging to the crown were selected. The analysis of their temporal profile allowed verifying the regular behaviour of the structure. The temporal profiles are plotted in Fig. 3 and it is worthwhile noting that P1, which is located close to the area where the dam is bound to the mountain, remains stable for the entire lapse. By contrast, P8, P9 and P14, which are in the central part, suffer from a displacement up to 4 mm. P6 occupies an intermediate position and has a smaller displacement.

We recall that a negative displacement means an approaching to the radar.

During the GB SAR campaign the dam was under monitoring by a coordinatometer, whose results were compared to IBIS-L results. An exaple of this comparison is shown in Fig. 4

Differences between measurements achieved by different instrumentations during 24 hours have been compared, resulting in discrepancies with a RMSE of ±0.2 mm (see Fig. 4).

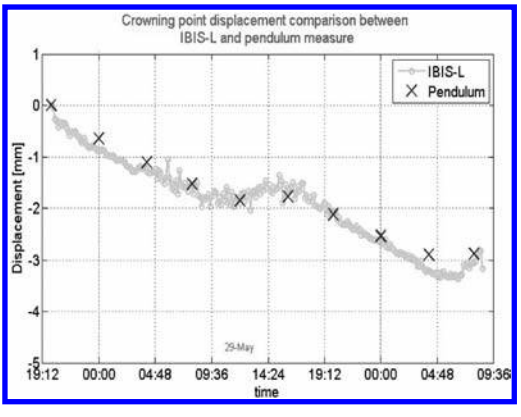

Figure 4. Comparison between displacements measured through a pendulum an the corresponding values retrieved from GB SAR measurements as a function of the time (after Alba et al., 2008).

4.2 *RAR*

In this section the potential of the RAR approach is discussed showing the results of the monitoring of two different structures located near Barcelona: the Collserola communications tower, designed by the famous architect Norman Foster and built in 1991 for the 1992 Summer Olympics, and a building in the city of Gavà close to the C31 highway. The aim is to observe the oscillations of these structures, determine their displacement at different points and analyze its frequency behaviour.

4.2.1 *The Collserola tower*

The tower was monitored on October 2, 2008, through the IBIS-S RAR available at the Institute of Geomatics, installed close to the base with an observation LOS of 67° with respect to the horizontal direction. The radar parameters were set to obtain a maximum range of 400 m with a range resolution of 0.5 m and a maximum measurable oscillating frequency of 53.845 Hz. The structure (located at Latitude 41° 25′ 02.00″ N, Longitude 02° 06′ 51.40″ E) stands from an altitude of 445 m asl, reaching a maximum 288 m height.

The weather conditions during the experiment indicated an air temperature of 14.1°C and a regular wind with strength of 6.6 m/s and a direction of 229°.

The position of the instrument was changed to observe different directions for the same points. Fig. 5 shows a photo of the RAR and the tower and a scheme of the measurement geometry, explaining the relationships between radar bins and the corresponding reflecting points of the structure.

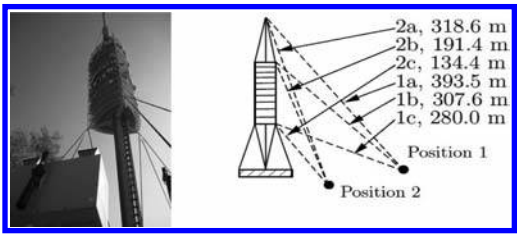

Figure 5. Left: view of the RAR and of the tower. Right: location of the different radar bin.

The hypothesis is that the tower, stimulated by the wind, oscillates close to its natural frequency. In Fig. 6 the measured displacement profile of different points is shown, identified though their corresponding radar bins, as a function of time. Two effects can be appreciated in the Fig. 6: the harmonic behaviour of the structure through a neat sinusoidal trend for all the points, and the variation of the amplitude which becomes larger as the distance from the base of the tower increases, as expected from the simple model of a clamped/free bar.

The oscillating frequency can be estimated for any observed point applying a frequency analysis to a single radar bin. The RAR, when correctly installed, is then able to provide spatial and temporal monitoring.

4.2.2 *The Gavà building*

The monitoring of a building located in the town of Gavà has also been carried out to evaluate the capability of the RAR in a different situation. The block of flats is 30 m high and the observation LOS of 45º with respect to the horizontal direction. The maximum distance was 150 m, with a range resolution of 0.5 m. The sampling frequency was 97.04 Hz. The measurement duration was 15 minutes.

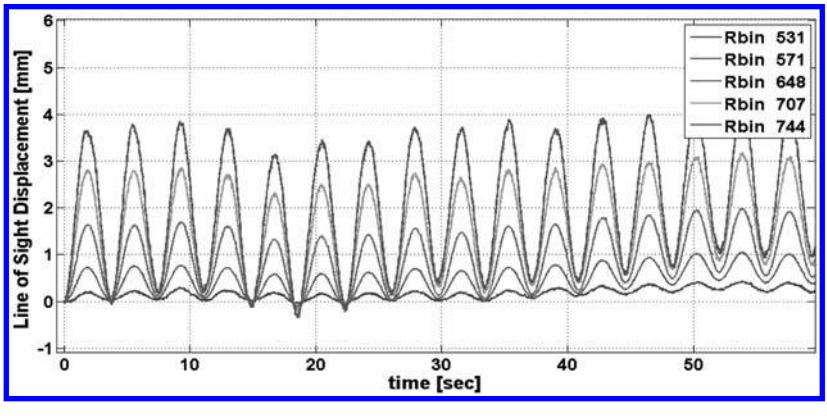

Figure 6. Time series of displacements measured from the first position at Collserola site. Different colours refer to different radar bins; the figure identifying the radar bin increases as its distance from the RAR.

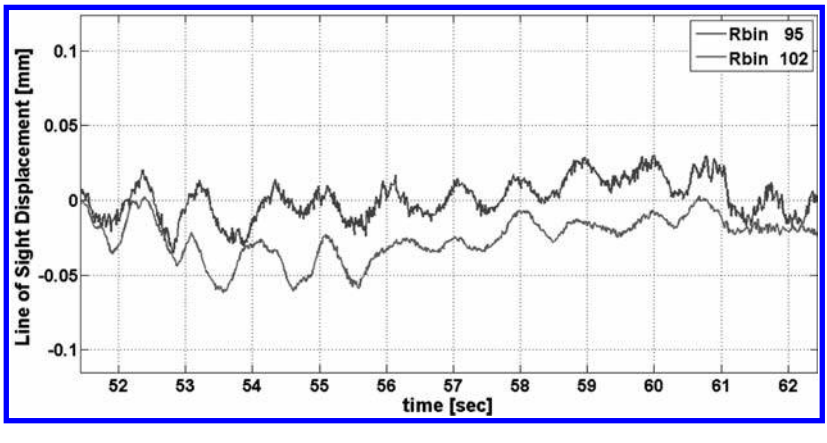

Figure 7. Time series of displacements measured at two positions of the Gava building. Different colours refer to two different radar bins.

In this case, the different characteristics of the structure resulted in more difficult observations, and smaller amplitudes of vibration were expected as confirmed from the time series shown in Fig. 7. This test confirmed that the high sensitivity and precision of the technique are undisputed: a less than ±50 μm peak to peak amplitude of displacement is measured.

5 CONCLUSIONS

The two ground based radar techniques, RAR and SAR, tested for the monitoring of dam and more generally the deformations of man-made objects and infrastructures appeared very promising. The 2D imaging sensor based on the SAR technique can provide millimetre precision with fine spatial details over the entire surface of the observed area. When dynamical information as oscillating frequency are required, the RAR approach demonstrated to be adequate and reliable. In addition, this type of measurement could potentially offer an interesting input for the structural analysis (and its monitoring over time) of dams.

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Other related themes

Dam Maintenance and Rehabilitation II – Romeo García et al. (eds)
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Maintenance and rehabilitation issues of the hydropower plants of the Drin and Mat River Cascades in Albania

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ABSTRACT: The three hydropower plants Fierze, Koman and Vau I Dejes located on the Drin River Cascade account for over 85% of electricity production in Albania. These plants are owned by the Albanian Power Corporation (KESH). A "Dam Safety Survey for Hydropower Plants Located on the Drin and Mat River Cascades" was completed in July 2006 with financing from the Swiss Government. The survey identified the following hazards and deficiencies: (i) natural hazards, such as rockfalls, floods, earthquakes; (ii) compliance with internationally recognized guidelines related to flood and earthquake safety; (iii) maintenance of dams and hydro-mechanical equipment; (iv) dam monitoring systems and analysis of monitoring data; (v) dam safety organisation within KESH; (vi) emergency action plans; and (vii) training of personnel. A series of recommendations with priority order were made based on an integrated dam safety concept. The safety evaluation carried out for these important dam projects is discussed.

1 INTRODUCTION

A survey of the present state of the safety of the three hydropower plants of the Drin River Cascade (Fierze, Koman, Vau I Dejes) and the two hydropower plants of the Mat River Cascade (Ulza, Shkopeti) in the northeastern part of Albania was carried out. In this context, the dams and appurtenant structures were inspected to identify the main hazards, and the present conditions of the civil structures and the hydromechanical equipment were assessed based on the visual observations and the available data.

The primary objective of this survey was to promote good engineering practice and modern technology in the field of dam safety, bearing in mind the possibly disastrous consequences of any dam failure. Dam safety is a fundamental prerequisite for sustainability of hydropower generation. Without adequate safety, not only are there large numbers of people at risk, but the power production will be jeopardized. Since the Drin and Mat River Cascades represent about 85% of the total electricity production in Albania, any loss of power production will have a dismal impact on the economy.

Dam safety is an integral concept that includes the following items:

- Structural safety of dam (assessment of site and dam type; design against actions from natural and man made environment), and technical information and documentation;
- Dam safety monitoring (visual and instrumental observations, periodic dam safety evaluations, etc.);
- Operational safety and dam maintenance (reservoir rule curves, qualified staff, etc.); and
- Emergency planning (emergency action plans, alarm system, etc.).

The investigations carried out were concerned with all of these items. The status of the structural safety of the dams built along the Drin and Mat rivers several decades ago was uncertain, because the design concepts and the design criteria for hydrological and seismic safety have undergone considerable changes. Moreover, much of the know-how in dam engineering in Albania has been lost due to the organizational and political changes. Therefore, in the first step, the safety of these dams was assessed, and remedial actions were proposed, where necessary, to reduce the most urgent risks. The most important observations, conclusions and recommendations of this study are discussed in the subsequent sections.

2 TECHNICAL DATA OF DAM PROJECTS

2.1 *Drin River Cascade*

The Drin cascade consists of Fierze dam (Table 1), the uppermost dam of the Drin cascade, Koman dam (Table 2) downstream of Fierze dam and the

Table 1. Fierze earth core rockfill dam.

Construction period	1971–78
Dam height (m)	177
Crest length (m)	380
Dam volume (million m^3)	8
Annual inflow (million m^3)	6370
Active storage (million m^3)	2300
Discharge capacity (m^3/s)	2600
Installed power	4 × 125 MW

Table 2. Koman concrete face rockfill dam.

Construction period	1980–85
Dam height (m)	115
Crest length (m)	250
Dam volume (million m^3)	5
Annual inflow (million m^3)	9114
Active storage (million m^3)	200
Discharge capacity (m^3/s)	3300
Installed power	4 × 150 MW

Figure 1. Fierze dam.

Vau I Dejes reservoir downstream of Koman dam. Vau I Dejes reservoir is formed by three dams, i.e. Qyrsaq rockfill dam with clay core and integrated concrete spillway and intake structure (Table 3), Zadeje rockfill dam with clay core, and Ragam embankment dam with clay core.

2.2 *Mat River Cascade*

The Mat River Cascade consists of two hydro power plants, i.e. Ulza (Table 4) and Shkopeti (Table 5).

Table 3. Qyrsaq rockfill dam with clay core.

Construction period	1967–71
Dam height (m)	46
Crest length (m)	375
Dam volume (million m^3)	1.2
Annual inflow (million m^3)	9460
Active storage (million m^3)	250
Discharge capacity (m^3/s)	4200
Installed power	5 × 100 MW

Table 4. Ulza concrete gravity dam.

Construction period	1952–58
Dam height (m)	64
Crest length (m)	260
Dam volume (m^3)	260,000
Annual inflow (million m^3)	1168
Active storage (million m^3)	240
Spillway capacity (m^3/s)	2160
Installed power	4 × 6.25 MW

Table 5. Shkopeti concrete arch-gravity dam.

Construction period	1958–63
Dam height (m)	52
Crest length (m)	88
Annual Inflow (million m^3)	1168
Active Storage (million m^3)	40
Spillway capacity (m^3/s)	5250
Installed power	2 × 12.5 MW

3 INSTITUTIONAL DAM SAFETY CULTURE

The following recommendations are made for the establishment of an institutional dam safety culture:

1. It is assumed by the general public and non-expert decision makers that a well-designed dam is safe and remains safe throughout its economic life. It is forgotten that the economic life very much depends on the maintenance of a dam. It is a fact that the life of a dam is as long as proper maintenance is guaranteed. If a major dam is neglected or abandoned, its safety deteriorates very fast. It must also be kept in mind that dam safety is the main prerequisite for the successful operation of a hydropower plant. Therefore, a comprehensive dam safety culture of all parties (i.e. dam engineers, dam owners, dam operators, dam safety authorities) involved

in the safety and operation of dams is extremely important.
2. A mechanism has to be found that ensures sustainability of the power production, which guarantees both dam safety and maximum power output, i.e. a mechanism where dam safety is also a primary objective.
3. The dam operators and owners must establish a dam safety system for each dam with clearly defined responsibilities and with a time schedule for periodic checks. The dam owners must have access to dam engineers and experts, either in-house or external, who can be mobilized at short notice in the case of problems.
4. A time schedule for regular dam safety checks and thorough dam safety assessments shall be established by the dam operator. Annual dam safety reports by the owner's engineers and detailed dam safety assessments by independent dam experts at five year intervals shall be prepared and submitted by the dam owners and operators to the dam safety authorities.

It has been found that there is a shortage of funding for maintenance, upgrade and repair, especially for civil and hydro-mechanical structures. Dams must be maintained to keep them in a safe condition. Occasional upgrade or rehabilitation is necessary due to deterioration, changing technical standards and improved techniques, increases in the downstream population, and changing land use. When the hazard classification of a dam is modified to reflect a higher hazard potential, the dam may need to be upgraded to meet an increased safety standard.

The lack of funding for dam upgrade is a serious problem. Unfortunately, operation, maintenance, and rehabilitation of dams can be very expensive. The owners are responsible for these expenses. However, many dam owners may not be able to afford these costs.

There is not only a shortage of funding on the side of the dam owners but also inadequate resources for the dam safety authorities.

Moreover, it was found that technical data on the hydropower plants of the Drin and Mat river cascades were not readily accessible or missing.

4 IDENTIFIED HAZARDS AND DEFICIENCIES

Based on site visits, the review of the obtained data, and the discussions held with KESH's staff the following hazards and deficiencies have been identified:

i. A number of natural hazards, such as rockfalls, faults located close to the dam foundations, floods, earthquakes, etc. threaten the safety of the dams.
ii. The structures designed in accordance to the prevailing hydraulic and seismic standards are deficient when today's internationally recognized guidelines are applied.
iii. The dams and the hydro-mechanical equipment have not been properly maintained. Inadequate budgets have been allocated for the periodic maintenance works.
iv. The dam monitoring systems are generally in an unsatisfactory condition. Most of the existing instruments are no longer working. Moreover, the recorded data are not evaluated regularly.

Based on the above observations, it is concluded that the present state of the dam safety is uncertain.

5 EMERGENCY CLASSIFICATION

As an aid for judging the level of severity an assessment matrix can be developed (which may change from one facility to another one, depending on the dam's characteristics and the environment) (Table 6). Upon discovery of, or after having been notified about, an unusual scenario, it must be judged, whether external assistance is needed and whether there are adverse impacts with a threat to population, structures or environment. The urgency of the situation is the major factor in classifying the severity of an incident.

Table 6. Example of hazard matrix for hydropower plant showing hazards and required protective measures (Emergency classification: A: internal alert; B: developing situation; C: imminent situation), (Protective measures: (1) Rehabilitation, (2) Partial reservoir drawdown, (3) Full reservoir drawdown, (4) Evacuation, (5) Post-event evacuation).

	Protective measures				
Hazards	(1)	(2)	(3)	(4)	(5)
Natural hazards					
Floods	A	B		C	
Earthquake					C
Mass movements	A	B			
Structural hazards					
Spillway gates, equipment failure			C	C	
Differential movement of structure	A	B	C	C	
Embankment piping or seepage		B	C		
Electrical/mechanical failure	A				
Man-made hazards					
Sabotage, terrorism, acts of war		B			C

The following alarm levels and emergency situations can be distinguished:

i. The internal alert triggered by an unusual situation can be managed and controlled by the dam's staff. Typical internal alert scenarios are flood warning prior to receiving information on the size of the flood and potential dangers, and also abnormal monitoring results where readings on certain instruments exceed pre-set safety limits (e.g. piezometric heads, discharge from drainage facilities or displacement of structures).
ii. A developing situation exists when the observed incident clearly tends to turn into a serious threat to the dam's safety and the population in the downstream area. At this stage it is not yet known whether the situation can be brought under control.
iii. An imminent situation has developed when it has become clear that the progress of the incident or threat cannot be stopped but its consequences can still be mitigated, such as the evacuation of the population in danger.

6 HAZARDS AT DAM SITES

6.1 *General—flood hazard*

The main hazards from the natural environment to be considered for all dams are the flood hazard and the seismic hazard, as the design criteria have changed since the design of the dam. The analysis and design reports of all the dams were not available and it could not be checked how the dams were actually designed for these actions.

For the flood safety, the n-1 rule is applicable for embankment dams under the PMF. This is mainly a problem for Fierze and Koman dams as spillway is a gated tunnel spillway consisting of two tunnels. If one of these large capacity tunnels is out of order then the n-1 criterion cannot be satisfied.

6.2 *General—earthquake hazard*

The seismic design criteria and methods of seismic analysis have changed since 1989 when ICOLD Bulletin 72 (2010) was published. It must be assumed that the Albanian dams were designed against earthquakes using a seismic coefficient and the pseudo-static method of analysis. These concepts are considered outdated today. Therefore, a modern seismic hazard analysis had to be performed for all dam sites and based on these new seismic hazards a dynamic analysis of all embankment dams was performed for the safety evaluation earthquake.

As can be seen from Table 7, the Peak Ground Accelerations (PGA) at the dam sites vary from 0.42 g to 0.48 g for the horizontal earthquake component. The PGA values of the vertical component were taken as 2/3 of those of the horizontal component.

Table 7. Peak Ground Accelerations (PGA) on outcropping rock for Safety Evaluation Earthquake (SEE) at different dam sites.

HPP site	SEE (annual probability of exceedance 1/10,000)	
	horizontal (g)	vertical (g)
Fierze	0.416	0.277
Koman	0.448	0.300
Vau i Dejes	0.478	0.319
Ulza	0.435	0.290
Shkopeti	0.437	0.291

For the seismic analysis of the embankment dams a time history analysis was carried out using three different earthquakes. The horizontal and vertical components of the acceleration time histories were generated artificially by fitting the target response spectra shown in Fig. 2.

6.3 *Specific hazards at Fierze dam*

The main hazards from the natural environment at Fierze are (i) the Porava landslide, and (ii) the earthquake safety of the spillway intake tower No. 3.

6.3.1 *Impulse wave due to Porava landslide*

The freeboards at Fierze under normal and maximum reservoir levels are 16 m and 11 m, respectively. The freeboard is relatively large and was chosen due to the fact that the reservoir and the dam are threatened by a potential landslide named Porava slide that is located on the left bank of the reservoir about 1500 m upstream of the dam. An assessment of impulse waves generated by the Porava slide, reaching the Fierze Dam and running up on the opposite reservoir bank was carried out.

The Porava landslide developed in late Quaternary time during the formation of the Drin Valley. The landslide has a total volume of 34 million m^3 and about 50% of its volume is submerged by the Fierze reservoir. As the landslide is more than 10,000 years old, it has to be assumed that it has been subjected to strong ground shaking in the past. Moreover, the reservoir has been operated for some 30 years with annual fluctuations of the reservoir and no movements of the Porava landslide have been reported, therefore the landslide is considered

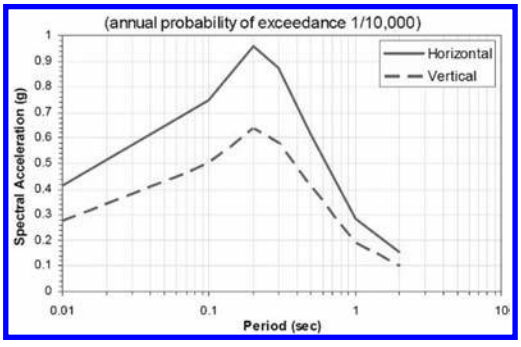

Figure 2. Acceleration response spectra for 5% damping obtained from the probabilistic seismic hazard analysis.

in a stable state under normal condition. However, in the case of a strong earthquake, extreme rainfall, sudden snowmelt or rapid reservoir draw down etc. the landslide may be reactivated. Therefore, a reassessment of the run-up elevation of an impulse wave in the reservoir generated by three different landslide scenarios was performed.

The analysis shows, that the impact velocity has a dominant influence on the results whereas the variation of the reservoir elevation has a comparatively small effect on the impulse wave height. It is evident that the maximum run-up height at the opposite bank of the reservoir is 10 to 20 times larger than at the Fierze dam. This fact can be explained by the different impulse directions. The opposite bank is situated in the main impulse direction of the slide whereas the wave propagating sideward and downstream the reservoir is attenuated. Moreover, due to the bend in the reservoir wave diffraction takes place which leads to a further reduction in the maximum wave height arriving at Fierze dam.

Assuming an impact velocity of 20 m/s, a diffraction coefficient of 0.6 a reservoir elevation with a freeboard of 11 m, then the maximum run-up height reaches up to 4.6 m. According to this calculation there is a freeboard reserve of 6.4 m.

6.3.2 *Earthquake safety of spillway intake tower*

After the Safety Evaluation Earthquake (SEE), which represents the worst ground motion to be expected at the dam site, the spillways must function and it must be possible to release a flood with a return period of about 200 years to prevent overtopping of the dam.

The seismic safety evaluation of the spillway intake tower (Fig. 3) was carried out based on the results of the nonlinear time history analysis of a cantilever beam model of the spillway intake tower (three-dimensional model) subjected to the SEE ground motions.

It was found that the gate room structure on top of the intake tower will be severely damaged and has to be replaced. According to the different linear and nonlinear time history analyses performed the peak acceleration on top of the intake tower varies from 38 to 40 m/s^2 in the two horizontal directions and up to 10 m/s^2 in vertical direction. The floor response spectra determined on the top of the intake tower serve as the basis for the seismic design of the new gate room structure.

In the assessment of the earthquake safety of the spillway intake tower reference is made to the large intake towers of the Zipingpu concrete face rockfill dam (156 m high dam) in China (Fig. 3), which were located in the epicentral region and experienced strong ground shaking during the May 12, 2008 Wenchuan earthquake (Fig. 4).

As these towers are quite similar to that of Fierze dam and since mainly the gate room structures on top of the towers suffered major structural damage, whereas the reinforced concrete towers did not show any visible signs of damage, the proposed rehabilitation of the spillway intake structure of Fierze dam is mainly concerned with the gate room structure. As the intake tower at Fierze will also be damaged based on the results of the dynamic analyses and therefore it should also be rehabilitated, however, this would be a major and costly task.

As the spillway capacity at Fierze is not adequate additional spillway capacity must be provided in the future. By this extra spillway capacity there is no need that the spillway tower is fully functional as a moderate flood can still be released after the SEE by the other spillway openings. Therefore it is proposed not to strengthen the existing intake tower (spillway No. 3) in Fierze at this time.

6.3.3 *Erosion in plunge pool of Fierze spillway*

The national highway is crossing the Drin river immediately downstream of the plunge pool of

Figure 3. Spillway intake tower at Fierze with gate room structure on top.

Figure 4. Damage of gate structure on top of intake tower of Zipingpu CFRD caused by the Wenchuan earthquake, China.

Figure 5. Erosion of abutments of national highway at Fierze dam due to spillway operation (overtopping of bridge and debris in the river).

the Fierze spillway. When the spillway is in operation substantial scouring and erosion of the bridge abutment takes place and the bridge has been overtopped during recent floods in early January 2010 (Fig. 5).

It is obvious that this bridge cannot withstand the effects of major floods and therefore alternate bridge locations have to be investigated.

6.4 *Erosion in plunge pool of Koman spillway*

The operation of the spillway of Koman dam has caused extensive river bank erosion and the toe wall of the dam founded on alluvial soil has been destroyed and parts of the dam toe have been eroded (Fig. 6). River bank protection and protection of the dam toe is needed. During the severe floods in early January 2010 the right river bank suffered additional damage.

6.5 *Sedimentation in Koman reservoir*

The Koman reservoir is used for navigation from Koman to Fierze. There is a regular ferry service, which can transport passenger cars and heavy trucks. The Koman reservoir is very narrow and some tributaries transport large amounts of sediments into the Koman reservoir.

At these locations the water depth is getting shallow and at lower water level the ferries cannot operate. Therefore, periodic dredging of the sediments is needed at such critical locations.

6.6 *Rockfall hazard at Shkopeti and Koman dams*

On both abutments of Shkopeti and Koman dams there are almost vertical cliffs with potentially unstable rock masses, which could damage the dam crest, spillway (Shkopeti) and/or intake structures during a strong earthquake. Different measures are planned to stabilize the rock masses and to protect parts of the dam crest (Fig. 8).

Figure 6. Erosion of dam toe due to spillway operation at Koman.

6.7 *Deterioration of hydro-mechanical components*

Some of the spillway intake gates have not been in operation for several years and it is not clear if they can be operated as originally planned in the case of floods. Also due to deformational problems with stoplogs it is not possible to inspect the condition of one important radial gate located in the middle of the spillway tunnel of Koman. Moreover, the corrosion protection of steel works is deteriorating and some of the seals of the spillway gates at Vau I Dejes are leaking (Fig. 9).

Figure 7. Sediments from tributary of Koman reservoir.

Figure 8. Rockfall protective structure at left bank of Koman dam.

Figure 9. Corrosion of penstocks and leakage of spillway gates.

6.8 *Dam instrumentation and dam safety monitoring*

Most of the instruments used for dam safety monitoring do not comply with today's standard and have to be replaced. Also the recorded data are not evaluated systematically as discussed in Section 3. The upgrading of the dam monitoring system is under way.

6.9 *Miscellaneous items*

There are a number of other items, which have to be addressed as some of the components have reached their service life. In the past the emphasis has been on the maintenance of the electricity generating equipment and other items have received lower priority as the economical benefits of maintenance and dam safety works are not readily recognizable. They are considered as expenses with no or little direct benefit. Here it is important that the dam owner takes a broader view on the benefits and provides an adequate budget for maintenance so the ageing process of the dams does not get out of control.

7 RECOMMENDATIONS

The following recommendations are made based on an integrated dam safety concept:

i. In view of the large scours observed at the Fierze and Koman dams, measures have to be taken to stabilize the plunge pools and to protect the dam toe at Koman.
ii. The operation of the cascades should be based on realistic rule curves, as well as well-defined guidelines for the safe operation of the dams in emergency situations. The floods must be released from the reservoirs in a safe and controlled manner with the objective of minimizing flooding in the populated downstream valley.
iii. The defective hydro-mechanical components must be repaired, and the obsolete devices must be replaced.
iv. The dam operators and owners must establish a dam safety system for each dam with clearly defined responsibilities and with a time schedule for periodic checks.
v. The detailed safety assessment of each dam shall be made by independent dam experts at say 5-year intervals and the corresponding report submitted for review to the Albanian dam authorities.
vi. The economical life of a dam is as long as regular maintenance can be guaranteed. Therefore, regular maintenance of all components of the dam is a vital task for the dam owner. Maintenance procedures shall be established for each dam. The necessary means and budget have to be made available.

vii. The monitoring system at each dam must be rehabilitated by repairing the existing instruments and installing additional instruments. The monitoring records must be systematically kept and analyzed.
viii. The engineers responsible for the dam safety at each plant should have the appropriate background.

8 CONCLUDING REMARKS

It is very important to develop a comprehensive dam safety culture among the dam engineers, owners, operators, and safety authorities. Although sufficiently comprehensive laws and regulations concerning dam safety already exist in Albania; a dam safety culture, like in most European countries, has not yet been established.

As an immediate measure, the maintenance of the hydropower plants, including all civil works (dam, appurtenant structures, access roads, etc.) and hydro-mechanical equipment, must be substantially improved. The negative psychological impression any signs of deterioration or ageing give to the public should not be underestimated.

No doubt, satisfactory maintenance needs sufficient budget and dedicated staff. Good maintenance and smoothly operating power plants provide positive motivation to the staff. It should also be noted that frequent rotation of staff and management could be detrimental to the long-term objective of safety of the dams and the hydropower plants.

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Estudio de la puesta fuera de servicio de las presas de Santa María de Belsué y Cienfuens, T.M. Nueno (Huesca), para la adecuación al RTSPE

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RESUMEN: El artículo describe el procedimiento de la puesta fuera de servicio de las presas de Santa María de Belsué y Cienfuens debido a la próxima puesta en servicio de la presa de Montearagón situada aguas abajo de éstas. Dichas presas dejarán de ser necesarias para el sistema de regadíos derivados del río Flumen, aunque puedan ejercer como laminadoras de caudales aún estando fuera de servicio. No obstante, no parece prudente mantener dichas infraestructuras en el estado actual de funcionamiento. Los trabajos consistirán en la elaboración de un proyecto que recoja las actuaciones necesarias para la puesta fuera de servicio de ambas presas según se establece en el RTSPE (Art.35) o en la Guía Técnica de Seguridad de Presas.

ABSTRACT: The article describes the process of decommissioning of Santa Maria de Belsué and Cienfuens dams due to the forthcoming commissioning of the Montearagón dam located downstream. These dams will no longer be necessary for the irrigation system from the river Flumen, although they can help in controlling the force of flood waters even while off duty. However, it seems imprudent to keep those infrastructures in the current state of operation. The work will include the drawing up of a project to collect the necessary actions for decommissioning of the dams as defined in RTSPE (Art.35) or in the Technical Guide for dam safety.

1 INTRODUCCIÓN

Por orden del Ministerio de Medio Ambiente de 12 de marzo de 1996 fue aprobado el Reglamento Técnico sobre Seguridad de Presas y Embalses (RTSPE), en el cual se fijan los criterios de seguridad que han de tenerse en cuenta para prevenir y limitar social y ambientalmente los riesgos potenciales que estas infraestructuras pueden presentar.

El RTSPE incluye en su ámbito de aplicación todas las fases de desarrollo y utilización de las presas y de los embalses, desde la fase de proyecto hasta la de eventual puesta fuera de servicio, cubriendo de esta forma una laguna muy importante existente en la legislación anterior.

En el presente estudio se definen los trabajos necesarios para la redacción del "Proyecto de puesta fuera de servicio de las presas de Santa María de Belsué y de Cienfuens para adecuación al Reglamento Técnico sobre Seguridad de Presas y Embalses", cuya titularidad corresponde al Ministerio de Medio Ambiente.

El proyecto se ajustará a la normativa vigente en cumplimiento al RD 9/2008 de 11 de enero de 2008, en concreto con el art.364 (3c), por el cual se deberá aprobar la Norma Técnica de Seguridad para la explotación, revisiones de seguridad y puesta fuera de servicio de presas.

Debido a la próxima puesta en servicio del embalse de Montearagón, aguas abajo de las presas de Santa María de Belsué y Cienfuens, éstas dejarán de ser necesarias para el sistema de regadíos derivados del río Flumen, aunque puedan ejercer como laminadoras de caudales aún estando fuera de servicio.

Tanto la presa de Santa María de Belsué como la de Cienfuens, situadas en la cabecera del río Flumen (T.M. Nueno. Huesca), fueron construidas para regular las aportaciones de cabecera y abastecer los antiguos regadíos del Flumen. Fueron terminadas en el año 1931.

Las presas de Belsué y Cienfuens producen un efecto laminador como presas de cabecera dignas de elogio, pues retienen los caudales de lluvias

torrenciales y los devuelven al cauce de forma progresiva. No obstante, no parece prudente mantener dichas infraestructuras en el estado actual de funcionamiento.

En el caso de Belsué, presenta un escenario incomparable para el estudio de la ingeniería de ríos, en cuanto a estudio de formación de meandro y llanuras de inundación, cada vez que se presenta una avenida al tener alternativas continuas de aguas altas y bajas.

El estudio define los trabajos de elaboración de un proyecto que recoja las actuaciones necesarias para la puesta fuera de servicio de ambas presas según se establece en el R.T.S.P.E. (Art.35) o en la Guía Técnica de Seguridad de Presas.

El proyecto de puesta fuera de servicio de las presas deberá incluir:

- Estudio de la estabilidad de todas las partes permanentes.
- Estudio de actuaciones sobre los elementos de las presas.
- Estudio geomorfológico.
- Evaluación de las consecuencias hidráulicas e hidrológicas.
- Estudio de adecuación de los accesos.
- Consecuencias de la nueva situación sobre otros usuarios, con especial atención a la planificación de las emergencias. Estas afecciones incluyen las consideraciones de tipo medioambiental que formarán parte del proyecto.
- Estudio de la Seguridad. Los trabajos para la puesta fuera de servicio necesitan autorización previa.
- Estudio de Seguridad y Salud.

2 DESCRIPCIÓN DE LAS PRESAS

2.1 *Presa de Santa María de Belsué*

La presa de Santa María de Belsué se encuentra en cabecera del río Flumen, en el término municipal de Nueno, en el norte de la provincia de Huesca. Se construyó para regular las aportaciones de cabecera y abastecer los antiguos regadíos del Flumen.

La presa de Santa María de Belsué es de gravedad con mampostería de planta curva de 183,00 metros de radio, con talud vertical aguas arriba y de 0,85 aguas abajo.

La capacidad de embalse es de 13 hm^3. La superficie de la cuenca es de 190,00 Km^2, con una superficie inundada de 82,75 ha. y una superficie regable mejorada de 2.217,00 ha. Se terminó de construir en el año 1931.

Tiene 52,00 metros de altura sobre cimientos, 126,61 metros de longitud de coronación a la cota 896,40 (m.s.m.). La cota de nivel máximo normal es la 894,40 m.s.n.m.

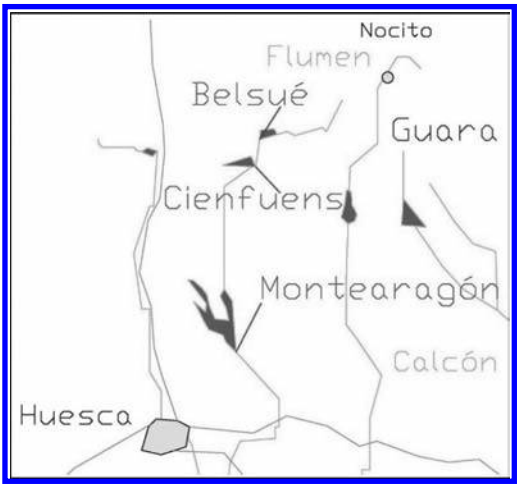

Figura 1. Plano de situación de las presas.

Figura 2. Vista aérea de las presas.

Figura 3. Vista aguas arriba de la presa de Santa Mª de Belsué.

Tiene previsto un aliviadero lateral (sin terminar) (130,00 m^3/seg) en el estribo izquierdo del cual únicamente se ha realizado parcialmente la excavación de la zona de vertido y del canal de descarga.

Figura 4. Vista aguas abajo de la presa de Santa Mª de Belsué.

Dispone también de una Toma Intermedia para 7 m^3/seg y dos Desagües de Fondo con capacidad de evacuación conjunta de 58 m^3/seg.

La Toma y los Desagües de Fondo no se han maniobrado desde hace más de 30 años, por lo que su funcionalidad no está evaluada.

2.2 *Presa de Cienfuens*

La presa de Cienfuens se encuentra situada en la zona de cabecera del río Flumen, a 2 kms aguas abajo del Embalse de Santa María de Belsué y a unos 7 kms aguas arriba del denominado Salto de Roldán. El nombre procede de las numerosas fuentes que en la zona existían debido a las filtraciones a través del Karst existente en todo este tramo del río.

La presa de Cienfuens es de gravedad de 33,70 metros de altura sobre cimientos con el talud de aguas arriba vertical y el de aguas abajo de 0,75 en forma escalonada.

En planta, es mixta, con forma circular en el centro de 70,00 metros de radio y rectos los dos estribos. La coronación mide 105,50 metros y está situada a la cota 851,40. La cota de máximo embalse normal es la 850,50 m.

La capacidad el embalse creado es de 1 hm^3. La superficie de la cuenca es de 192,00 km^2, con una superficie inundada de 8,70 ha.

El aliviadero es de labio fijo, situado en el extremo del estribo izquierdo descendiendo por la ladera por un pequeño canal excavado en la roca hasta verter al cauce. Dispone de dos desagües de fondo gemelos y una Toma Intermedia para el servicio de riegos. La superficie regable mejorada de 2.217,00 ha. Se terminó de construir en el año 1931.

2.3 *Presa de Montearagón*

La Presa de Montearagón es de hormigón vibrado, con una cota máximo embalse normal de 579,50 m.

Figura 5. Vista aguas arriba de la presa de Cienfuens.

Figura 6. Vista aguas abajo de la presa de Cienfuens.

Como órganos de desagüe existen un aliviadero de labio fijo de 45 m (506,00 m^3/s) y un desagüe de fondo formado por dos conductos de $1 \times 1{,}1$ m (60,6 m^3/s). La longitud de coronación es de 338,18 m. La superficie de la cuenca es de 144,00 km^2 y la superficie inundada es de 203,40 ha.

La Presa de Montearagón, creará un embalse de 51,5 hm^3 de capacidad que, al regular los caudales del río Flumen cuya aportación media es de 45,10 hm^3, hará posible el riego de 6.350 ha con una demanda de 31,9 hm^3, el abastecimiento a Huesca (hasta una demanda anual de 5 hm^3, una población abastecida de 50.000 hab) y el caudal de mantenimiento del río con una demanda anual de 5 hm^3. Además será viable la mejora en el control y laminación de avenidas y otros posibles usos.

3 OBJETO DEL ESTUDIO

El presente estudio, en concordancia con lo establecido en la Instrucción para el Proyecto, Construcción y Explotación de Grandes Presas, publicada en el B.O.E. de 27 de octubre de 1967, en la Directriz Básica de Planificación de Protección Civil ante el riesgo de Inundaciones, publicada

en el B.O.E. de 14 de febrero de 1995, y en el Reglamento Técnico sobre Seguridad de Presas y Embalses (RTSPE), publicado en el B.O.E. de 30 de marzo de 1996, propone las condiciones que deberán regir la redacción del "Proyecto de puesta fuera de servicio de las presas de Santa María de Belsué y Cienfuens. T.M. Nueno (Huesca), para adecuación al RTSPE".

4 DESCRIPCIÓN DE LOS TRABAJOS A REALIZAR

El estudio recoge las actuaciones necesarias para la redacción de un proyecto de puesta fuera de servicio de ambas presas según se establece en el RTSPE (Art.35) o en la Guía Técnica de Seguridad de Presas.

Es de prever que durante las obras de puesta fuera de servicio se afecte a la regulación del propio río aguas abajo de ambas presas, aspecto que deberá analizarse para proyectar las medidas necesarias que eviten tal circunstancia en la medida posible.

El proyecto de puesta fuera de servicio de las presas, deberá incluir:

- Estudio de la estabilidad de todas las partes permanentes.
- Estudio de actuaciones sobre los elementos de las presas.
- Estudio geomorfológico.
- Evaluación de las consecuencias hidráulicas e hidrológicas, y en particular un estudio específico del embalse vacío, con las consecuentes variaciones en las características del cauce, el efecto de las avenidas y estiajes en el valle de aguas abajo y el posible transporte de sedimentos procedentes de los depósitos que se hubieran creado en el embalse.
- Estudio de adecuación de los accesos. Se tratará la posible revegetación del embalse así como el estudio particular de accesos para la solución adoptada.
- Consecuencias de la nueva situación sobre otros usuarios, con especial atención a la planificación de las emergencias. Estas afecciones incluyen las consideraciones de tipo medioambiental que formarán parte del proyecto.
- Estudio de la Seguridad. Los trabajos para la puesta fuera de servicio necesitan autorización previa y el desmontaje total o parcial de los equipos o elementos estructurales de la presa no deberá iniciarse hasta que el embalse esté vacío, salvo que otras disposiciones adicionales garanticen su permanencia en niveles bajos y la seguridad de las obras que puedan ponerse en carga.
- Estudio de Seguridad y Salud.

A continuación se resumen los trabajos necesarios que deberán realizarse, con independencia de otras actividades complementarias:

4.1 *Fase I: Estudio de antecedentes*

El estudio plantea en una primera fase un estudio de antecedentes recopilando la información existente en la Confederación Hidrográfica del Ebro relacionada con las actuaciones a realizar en las presas de Santa María de Belsué y Cienfuens.

También se recogerán todos los proyectos, planos y demás documentos técnicos relacionados con las infraestructuras sobre las que se proyecta la actuación.

De cada una de las actuaciones se realizará un breve documento de síntesis, en el que se reflejen apoyándose en la pertinente documentación gráfica los puntos principales de la realización.

Las principales actuaciones serán las siguientes:

- Estudio y Recopilación de Datos.
- Recopilación de información básica.
- Análisis de los condicionantes de la intervención.
- Inventario de infraestructuras.
- Inventario de bienes de interés cultural, arquitectónico y de ocio.
- Recopilación de Estudios previos hidrológico-hidráulicos.
- Documentación Ambiental.
- Recopilación de cartografía y topografía básicas.
- Relación e inventario de bienes y servicios afectados.

4.2 *Fase II: Análisis y definición de las soluciones*

La fase de análisis y definición de soluciones será la que caracterice los trabajos específicos a realizar en cada una de las actuaciones. En general estos trabajos precisarán de una toma de datos y de una realización de cálculos y estudios de soluciones.

Como paso previo al estudio de alternativas, deberán adquirirse en campo todos los datos necesarios que permitan la posterior realización de cálculos, el estudio y la selección de las soluciones más adecuadas.

Como elemento imprescindible en el estudio de soluciones habrá que considerar la forma de dar continuidad al río Flumen para que las obras no afecten a su explotación y la capacidad de dar servicio aguas abajo de la presa.

En los apartados siguientes se explican las condiciones a tener en cuenta en los trabajos a desarrollar. Estas condiciones sólo serán exigibles en el

caso de que la actuación precise de las actividades que, a continuación, se explican.

Las principales actuaciones serán las siguientes:

- Desarrollo de la solución propuesta.
- Visitas y Toma de Datos "in situ".
- Análisis de la seguridad de la presa y contraste de las actuaciones necesarias.
- Realización de Cálculos y Justificación de las Soluciones.
- Estudios de laminación de avenidas con las presas fuera de servicio.
- Uso de los medios de los embalses para usos educativos, de experimentación y/ó lúdicos.

• Cartografía y topografía

Dependiendo de la naturaleza de la actuación puede ser necesaria la utilización para los proyectos de cartografía y topografía.

• Reconocimiento de materiales y ensayos

En algunas actuaciones a realizar será necesario caracterizar los materiales del terreno, o de la propia presa para poder valorar la mejor solución del proyecto. Para ello se realizarán trabajos de campo en el emplazamiento de la actuación.

➢ Investigación de las características del terreno

Cuando no se disponga de una información previa suficiente se realizará una investigación de las características de los materiales naturales en la zona (laderas del embalse, de los accesos a la presa, estado del cimiento de presa, etc.).

Como complemento a los trabajos de cartografía geológico-geotécnica de superficie, se realizarán sondeos mecánicos con sus correspondientes ensayos geotécnicos en aquellos puntos en los que la definición de su geometría o estudio de su permeabilidad lo requieran.

En las zonas señaladas y con la frecuencia lógica para la definición de los materiales se podrán ejecutar:

Sondeos mecánicos
Calicatas
Geofísica: sondeos eléctricos verticales y sísmica de refracción.

➢ Investigación de las características de las presas

En primer lugar, se estudiará la documentación disponible que pudiese aportar alguna información sobre el estado actual de los materiales del cuerpo de la presa o de cualquier otro elemento de la misma. Si esta no fuese suficiente para definir la solución a adoptar se deberá realizar una campaña de reconocimientos que permita caracterizar los materiales.

La investigación a realizar se basará en la ejecución de sondeos mecánicos junto con los ensayos in situ y de laboratorio que permitan caracterizar los parámetros geotécnicos y resistentes de los distintos materiales que componen el cuerpo de presa y el cimiento.

En cualquiera de los dos casos anteriormente descritos, se realizará una propuesta de la campaña de investigación definiendo los siguientes aspectos como mínimo:

Perfiles de investigación y nº de sondeos en cada uno
Método de ejecución de sondeos
Ensayos a realizar in-situ
Ensayos a realizar en laboratorio
Ensayos necesarios para la determinación de las características dinámicas de los materiales

• Estudios geológicos y geotécnicos

Están muy relacionados con las investigaciones sobre materiales efectuadas en el apartado anterior. La geología a definir deberá ofrecer un nivel de conocimiento suficiente de la zona de la presa donde se vaya a realizar la actuación, de modo que constituya una base adecuada para la realización de los trabajos de mayor detalle asociados a cada elemento del proyecto. Así pues, si después de la recopilación y síntesis de la cartografía geológica de los trabajos previos ésta no fuese suficiente será preciso completarla con los planos geológicos que sean necesarios para la definición de la estructura de la zona.

En los casos en que sea preciso se realizará una cartografía geológica de detalle de las zonas conflictivas, destacándose los aspectos de mayor interés, como son el estudio de las condiciones de estabilidad, filtraciones, permeabilidad, naturaleza y espesor de los suelos, sistemas de fallas y juntas y características de los planos de discontinuidad. La información se presentará en planos y perfiles a escala adecuada.

Con los datos anteriores y los proporcionados por los ensayos se realizará un estudio que concluya con un informe que podría contener los siguientes aspectos, dependiendo de los casos:

Aspectos geotécnicos para conocer los condicionantes del terreno en actuaciones relacionadas con el cimiento de la presa, laderas del embalse, ejecución o adecuación de caminos de accesos a la presa, etc.

Conclusiones sobre las características de permeabilidad y resistencia a erosión por filtraciones, en actuaciones relacionadas con la mejora de estos aspectos en la presa.

Recomendaciones para las excavaciones necesarias para los caminos de acceso u obras de la propia presa, si fuese necesario. Incluirá la recomendación sobre los taludes, medios de excavación y medios adicionales para garantizar la estabilidad, en el caso de que se considere necesario.

Conclusiones sobre la estabilidad del conjunto presa-cimiento y su permeabilidad.

Recomendaciones sobre la cimentación del cuerpo de presa o aliviadero, con indicación, si fuese necesario de las técnicas y materiales a utilizar para garantizar la estabilidad del conjunto.

Recomendaciones sobre el tratamiento o mejora para la impermeabilización o mejora de la red de drenaje, con indicación del proceso constructivo, disposición de los taladros, orientación, estimación de permeabilidades y volúmenes de inyección.

Recomendaciones sobre el firme en actuaciones en caminos o en coronación de presa, indicando tipo, espesores de las capas y medidas de drenaje adicionales.

- Estudios hidrológicos

Los estudios hidrológicos son necesarios en aquellos casos en que la actuación consista en proyectar elementos de desagüe, capacidad del aliviadero o cauce aguas abajo, estabilidad del embalse ante determinados episodios de llenado-desembalse, etc. En los casos en que la documentación previa no aporte la información necesaria o no esté lo suficientemente actualizada, se realizarán cálculos hidrológicos, entre los que pueden destacar:

Estudio de laminación de avenidas con las presas fuera de servicio.

Uso de los medios de los embalses para usos educativos, de experimentación y/ó lúdicos.

Estudio de inundabilidad

➢ Estudio de laminación de avenidas con las presas fuera de servicio.

El objetivo de este estudio hidrológico es la estimación del régimen de avenidas, que permita obtener los caudales asociados a distintos periodos de retorno y las avenidas necesarias para realizar el estudio de inundabilidad si fuese preciso. En el estudio de laminación se desarrollarán los trabajos necesarios a partir del régimen de avenidas estimado, de la cartografía y de los estudios de Inundabilidad.

En la calibración del modelo hidrológico se tendrá en cuenta los datos proporcionados por las estaciones de aforo, hidrometeorológicas e incluso los datos históricos, así como publicaciones al respecto.

Los caudales a calcular dependerán de los periodos de retorno, por tanto, en actuaciones relacionadas con los órganos de desagüe se determinarán las de 1.000, 5.000 y 10.000 años, mientras que para actuaciones relacionadas con accesos u obras secundarias se calculará la avenida de 500 años de periodo de retorno.

➢ Uso de los medios de los embalses para usos educativos, de experimentación y/ó lúdicos.

Se estudiará el provecho posterior de ambos embalses una vez puestos fuera de servicio, para dotarlos tanto de la seguridad necesaria desde el punto de vista turístico (dado que se encuentran en el P.N. de la Sierra de Guara), como para el aprovechamiento ulterior en aspectos técnicos de estudio hidrológico (estudio de formación de meandros, etc …).

➢ Estudio de inundabilidad

Estos estudios sólo serán precisos en el caso de que sea necesario conocer la posible inundación ocasionada por una determinada avenida con el fin de adecuar el cauce aguas abajo de la presa o bien situar elementos accesorios, como caminos de acceso, edificios, etc. de forma que queden inundados ante determinadas ocasiones.

- Cálculos hidráulicos.

Algunas actuaciones a realizar podrían estar relacionadas con la mejora, o la adecuación al reglamento. En estos casos, siempre y cuando no haya información precedente al respecto, serán necesarios realizar los estudios hidráulicos necesarios para un correcto diseño. Así pues, en los casos en que sea necesario estos cálculos hidráulicos, se partirá de los hidrológicos y se tendrán en cuenta los siguientes aspectos:

- Cálculo que permita laminar avenida o riada, mediante la construcción de aliviadero, túneles u otras obras adicionales de desagüe.
- Cálculo de los órganos de rotura de carga previstas en ellas.

En todos los casos se comprobarán las condiciones y limitaciones que se establecen en la Instrucción y el Reglamento Técnico de Seguridad de Presas y Embalses.

- Cálculos estructurales.

Si el elemento, objeto de alguna de actuación (obras de paso de carreteras, canales, obras de drenaje, etc.), precisase para su correcto diseño unos cálculos estructurales, se procederá a la realización del mismo de forma justificada acorde con la las Instrucciones y Normas vigentes actualmente en España y en aquellos casos no previstos en estas Normas se justificarán debidamente las hipótesis de cálculo utilizadas. Se deberá concretar y describir los programas de cálculo de los que vaya a hacer uso.

Para cada estructura se definirán claramente los siguientes conceptos:

Metodología
Bases de cálculo
Cálculos
Resultados
Conclusiones.

Para el cuerpo de presa propiamente dicho (incluyendo el estribo izquierdo), los cálculos se realizarán teniendo en cuenta las solicitaciones

previstas en la Instrucción de Grandes Presas, el contenido del Reglamento Técnico de Presas y Embalses y las guías técnicas correspondientes.

Se incluirá, si fuese preciso, los siguientes estudios específicos:

Estado tensional, de deformación y desplazamientos del cuerpo de presa mediante modelos de elementos finitos

Estudio de estabilidad del cuerpo de presa y de los estribos

Estudio de estabilidad de taludes para superficies de deslizamiento circulares o mixtilíneas

Cálculo de redes de filtración en medios porosos (si la presa es de materiales sueltos)

Cálculos dinámicos si la sismicidad de la zona lo aconsejara.

- Auscultación e instrumentación.

No existe ningún tipo de aparato de medida. Sería necesario informar de la situación óptima para la ubicación de aforadores en el entorno de ambas presas para el control de caudales en el río Flumen.

- Instalaciones eléctricas.

Los estudios específicos de instalaciones eléctricas, engloban las actuaciones correspondientes a instalar o mejorar el sistema eléctrico de la presa. Así pues, las siguientes condiciones afectarán en el caso de que se requiera este tipo de actuaciones.

- Expropiaciones, servicios afectados y servidumbres.

En todas aquellas actuaciones en las que no se disponga de terrenos para poder afrontar las obras será preciso disponer de un proyecto de expropiaciones.

Se incluirá un plano en el que se señalarán los terrenos a expropiar, así como los servicios afectados (líneas de energía y comunicaciones) y servidumbres.

En el estudio de expropiaciones habrá que considerar los siguientes conceptos fundamentales:

A. Superficie de expropiación.
B. Base cartográfica.
C. Fuentes de información.
D. Relación concreta e individualizada de los bienes y derechos afectados por la expropiación.
E. Criterios de valoración.

A. Superficie de expropiación

Habrá de estudiarse con minuciosidad la delimitación de la superficie total que será necesaria expropiar para la ejecución de las obras.

B. Base cartográfica

Contendrá:

Plano a escala 1: 50.000 de toda la zona afectada por las obras.

Plano parcelario de dicha zona, obtenido:

Mediante levantamiento fotogramétrico existente o realizado a los mismos fines.

C. Fuentes de Información

Se utilizarán fundamentalmente las siguientes, sin perjuicio de la posibilidad de obtener datos de cualquier Organismo público, privado o de particulares:

Centro de Gestión Catastral y Cooperación Tributaria.

Ayuntamientos de los términos municipales afectados.

Cámaras agrarias.
Registro de la Propiedad.
Vecindario, prácticos …
Inspección directa.

D. Relación concreta e individualizada de los bienes y derechos afectados.

Habrá de contener los siguientes datos:

Nº de cada parcela en forma correlativa y topológicamente continúa.

Número de identificación catastral de cada parcela y del polígono y término municipal al que pertenezca.

Nombre, apellidos, NIF y domicilio de los propietarios y otros titulares de derechos afectados.

Superficie parcial que va a ser objeto de expropiación.

Calificación de terrenos por su tipo de uso rústico o su clasificación urbanística.

E. Criterios de valoración.

Se tendrán en cuenta criterios en que los baremos utilizados sean representativos del valor de mercado.

4.3 *Fase III: Redacción de proyecto*

Esta parte incluye como mínimo, los siguientes conceptos:

4.3.1 *Análisis medioambiental*

Cuando de acuerdo con la legislación vigente sea necesario, se realizará un Estudio de Evaluación de Impacto Ambiental de las obras proyectadas, de acuerdo con el contenido de la Ley 6/2001 de 18 de Mayo, o cualquier otra de los documentos técnicos que la legislación contempla al objeto de minimizar la afección al medio, estableciendo las medidas protectoras y correctoras.

Para este Estudio se tendrá en cuenta toda la legislación y normativa existente sobre espacios y patrimonio protegido, tanto a nivel nacional como autonómico o local.

En especial se tendrá en cuenta lo dispuesto en la Red Natura 2000, configurada como la red ecológica europea de Zonas Especiales de Conservación

(ZEC), y cuya creación viene establecida en la Directiva 92/43/CEE del Consejo, relativa a la conservación de hábitats naturales y de la fauna y flora silvestres, reconocida como Directiva Hábitats. El objeto de esta Directiva es contribuir a garantizar la biodiversidad mediante la conservación de los hábitats naturales y de la fauna y flora silvestres calificadas de interés comunitario, en el territorio europeo de los Estados miembros, mediante el mantenimiento o restablecimiento de los mismos en un estado de conservación favorable. En la Directiva se recoge expresamente que se integran en esta red las Zonas Especiales de Protección para Aves (ZEPA) ya clasificadas como tal o las que se clasifiquen en un futuro en virtud de la Directiva 79/409/CEE del Consejo, relativa a la conservación de las aves silvestres, conocida con Directiva Aves.

Este Estudio evaluará las alteraciones sociales, económicas y ambientales derivadas de la construcción y puesta en servicio de las obras proyectadas. También se evaluarán los efectos que puedan producirse sobre el patrimonio artístico.

Para ello se efectuará una previsión de los impactos que comportará la ejecución de las obras y, en función de éstos, se adoptarán las medidas correctoras adecuadas, con la finalidad de minimizar las alteraciones previstas, estableciéndose un Programa de Vigilancia Ambiental y Seguimiento.

De acuerdo con la finalidad señalada anteriormente, los objetivos serán los siguientes:

Analizar las características de las obras proyectadas desde el punto de vista constructivo, funcional y urbanístico.

Prever la naturaleza y magnitud de los impactos originados por el diseño de las obras proyectadas.

Establecer, en líneas generales, las medidas correctoras que, habiendo sido experimentadas técnicamente y siendo económicamente viables, minimicen los impactos anteriores.

Establecer un programa de vigilancia ambiental que permita asegurar el cumplimiento de las medidas correctoras recomendadas, así como el seguimiento del cumplimiento de las previsiones realizadas.

Se redactará un Documento de Síntesis en el que se recojan, en forma resumida, las conclusiones más importantes del Estudio.

4.3.2 *Estudio de seguridad y salud*

Se incluirá en el proyecto un estudio de Seguridad y Salud, en base a la normativa vigente, y en particular a las siguientes disposiciones:

Ley 31/1195, de 8 de noviembre, de Prevención de Riesgos Laborales (BOE nº 269 de 10 de noviembre de 1995).

Disposiciones Mínimas de Seguridad y Salud en los Lugares de Trabajo, aprobado por Real Decreto 486/1997, de 14 de abril (BOE nº 97 de 23 de abril de 1997).

Señalización de Seguridad y Salud en el Trabajo, aprobada por Real Decreto 485/1997, de 14 de abril (BOE nº 97 de 23 de abril de 1997).

Disposiciones Mínimas de Seguridad y Salud Relativas a la Utilización por los Trabajadores de Equipos de Protección Individual, aprobadas por Real Decreto 773/1997, de 30 de mayo. (BOE Nº 140 de 12 de junio de 1997).

Disposiciones Mínimas de Seguridad y Salud para la Utilización por los Trabajadores de los Equipos de Trabajo, aprobadas por Real Decreto 1.215/1997, de 18 de julio. (BOE Nº 188 de 7 de agosto de 1997).

Disposiciones Mínimas de Seguridad y Salud en Obras de Construcción, aprobadas por Real Decreto 1.627/1997, de 24 de octubre. (BOE Nº 256 de 25 de octubre de 1997).

Resto de Normativa en Materia de Seguridad y Salud vigente durante el período de duración de los trabajos.

4.3.3 *Estudio de soluciones*

Con el detalle necesario a nivel de proyecto, se realizará el estudio de posibles soluciones analizando cada uno de los aspectos referidos en el resto de los estudios específicos precedentes (Estudio de laminación de avenidas con las presas fuera de servicio, uso de los medios de los embalses para usos educativos, de experimentación y/ó lúdicos, estudio de inundabilidad, cálculos hidráulicos, estructurales, de estabilidad, geotécnicos, etc.).

Se estudiarán las obras relativas a estabilización de laderas, consolidación de estructuras, revestimientos, tratamientos de impermeabilización posibles mejoras en sistemas de drenaje, acondicionamiento de viales, nuevos accesos, etc.

Como elemento imprescindible en el estudio de soluciones habrá que considerar la forma de dar continuidad al río del Flumen.

A continuación se describen los puntos específicos a tener en cuenta en el estudio de soluciones de las diferentes actuaciones integrantes, así como el personal especializado que intervendrá.

5 CONCLUSIONES

Con la redacción del proyecto de puesta fuera de servicio de las presas de Santa María de Belsué y Cienfuens se lograría mejorar la seguridad en la regulación del río Flumen en su cabecera, ya que las actuales infraestructuras de ambas presas no cumplen con el reglamento de técnico de seguridad de presas y embalses. Su puesta fuera de servicio significaría la garantía en la laminación de caudales en la cabecera del río Flumen de un modo seguro, dejando su regulación a la nueva presa de Montearagón.

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Use of DMR (Dam Mass Ratio), a new geomechanics classification, derived from RMR, in safety appraisals of old dams' foundations

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ABSTRACT: Some topics, which difficult the effective use of RMR for dam foundations, are reviewed and a new geomechanics classification system, DMR (Dam Mass Rating), is proposed -as an adaptation of RMR- giving tentative guidelines for several practical aspects in dam engineering and for the appraisal of dam foundations in preliminary studies (overall stability against sliding, needed depth of excavation for the foundations, consolidation grouting treatment, possible consequences of excessive relationship -following Rocha concepts- between the deformation modulus of the dam and of the rock mass foundation) taking account of the effects of rock mass anisotropy and of the water saturation. The formulae for estimation of the rock mass deformation modulus E_m are also discussed. The DMR method can be useful when assessing the safety conditions of old dams, which are not well documented technically.

1 INTRODUCTION

Use of several geomechanics classifications is widespread in rock mechanics nowadays for tunnels, mining, slopes or, simply and very often, for rock mass characterization. Only the RMR (Rock Mass Rating) geomechanics classification (Bieniawski, 1973) was proposed for tunnels, slopes and foundations (specifically including sliding consideration of gravity dams' foundations).

A very interesting precedent in dam practice is the so called "Engevix preclassification" used in Itaipú to cope with the enormous amount of geotechnical data for the foundation of the long lateral wing dykes of the main dam. It was developed by Cruz (1976) and can be reviewed in Camargo et al (1978) and John (1978). Basically it is a rating system for the different properties of the rock mass attending to their effects in the dam foundation safety. Other interesting precedent is the Kikuchi (1979) classification, very well adapted to the geologically young volcanic terrains which prevail in Japan.

Use of RMR has been scarce in dam foundations (except for characterization purposes). Several authors have referred to it in actual projects (Di Salvo, 1982; Van Schalkwyk, 1982; Sánchez Sudon and Mañueco, 1991; Marcello et al, 1991; Hemmen, 2002 …). Pircher (1982) said that "the future seems to be in the development of quality index values e.g. RMR by Bieniawski" and Serafim (1988) stated that "appropriate rock mass classifications can … be used to obtain a good estimate of … parameters", both in General Reports for ICOLD Congresses on Large Dams.

Difficulties in RMR use for dam foundations derive from several points: consideration of the water pressure is very doubtful (the pore pressure ratio varies along the dam foundation, dams must operate with changing water levels …), there are no good rules for quantifying the adjusting factor for the joint orientation (which ideally should allow for the safety against total failure by horizontal shear, and/or for local failure), changes in properties of the rock, the rock mass and the joints are induced by watering changes (saturation, desiccation, flow along the joints …).

Bieniawski and Orr (1976) proposed guidelines for the RMR correction factor for general stability against horizontal sliding of gravity dams. General stability against horizontal sliding is a basic condition but not a very common problem (although there have been failures because lack of shear strength along joints, as in Malpasset).

An appraisal of the deformability of the rock mass is needed in order to calculate stresses, strains and deformations in dams. Hence, empirical correlations between geomechanics classifications and deformation modulus of the rock mass E_m have always been very popular. The first one of these correlations was proposed by Bieniawski (1978), and afterwards several authors have introduced modifications to improve it. Most of these correlations fail to consider changes due to two important aspects: anisotropy of rock mass and water effect.

The dam engineer needs, when comparing possible dam sites, rapid appraisals of several other topics:

- general adequacy of site conditions for the type of dam,
- depth of excavation of altered rock (if needed),
- required amount of foundation treatment (consolidation grouting),
- good deformability conditions of the site, appropriate for the type of dam.

So, there cannot be only an adjusting factor and a sole guideline. Besides conditions will be different according the type of dam (arch, gravity, rock fill with concrete face, embankment) and material (CVC, RCC, hard fill).

DMR (Dam Mass Ratio, Romana, 2003) is a geomechanics classification derived from RMR specifically for all dams' foundation: arch, gravity (conventional concrete, roller compacted concrete and hardfill) and embankment dams.

DMR classification can be applied both for preliminary studies of new dams and for safety appraisal of old dams with insufficient data registers on foundation properties. Many old dams which are in operation were designed by a combination of "good engineering sense" and experience. Actual information on their foundations is very scarce and doesn't follow state-of-the-art "good practice". DMR is a method for gathering geomechanics information about their foundations, allows safety preliminary appraisals and points to possible safety dangers in/because of the foundations. As all the geomechanics classifications, DMR is a useful tool for assessment of problems and possible solutions, but it is not a method of detailed project. DMR is being used actually in a number of countries (Argentina, Colombia, Iran, Portugal, Spain …).

The paper presents the last (2010) formulation of DMR geomechanics classification and gives guidelines for their use in old dams. Formula and tables have some differences with the prior formulations, differences derived from general practice of DMR.

2 DEFINITIONS

RMR includes 5 parameters and a correction factor according joints orientation respect the work (tunnel, slope and/or foundation). RMR_B (basic RMR) is defined as the addition of the 5 parameters, without the correction factor.

Hoek has advocated, speaking on the Hoek-Brown criteria, the use of a "dry RMR", obtained with the maximal rating of the water parameter, with simultaneous introduction of real pore pressures in the computations (see for instance the last version—"2002 edition"—in Hoek et al, 2002). No doubt that this is a sound procedure, when computing, and could be extended to geomechanics classification work.

RMR_{BD} (basic dry RMR) is defined as basic RMR in fully dry conditions (the last parameter, WR, related to water, is evaluated as 15).

For the purpose of DMR RMR_B parameters are evaluated as in the last version (Bieniawski, 1999). RMR parameters tables are well known and will not be repeated here, but an appendix to this paper gives some practical guidelines for classification field work in dam sites.

DMR_{STA} is the value of DMR related to dam stability

$$DMR_{STA} = RMR_{BD} + R_{STA}$$

where R_{STA} is the correction factor for joint orientation (dip) related to dam stability (Table 3).

DMRDEF is the value of DMR related to relative deformability, with WR = 5, and no correction factor for joints orientation. (Tab. 8).

3 INFLUENCE OF WATER ON BASIC RMR_B

The fifth parameter for calculation of RMR_B, WR, is related to water, with a relative weight on RMR_B up to 15 points (15% of the maximal total). Bieniawski offers three different methods to evaluate it: inflow of water in tunnel, flow across the joints and water pressure ratio.

The best method to determine this parameter in dams is the use of the water pressure ratio $r_u = u/\sigma_v$ were u is the water pressure and σ_v the total vertical stress in each point. The water rating WR can be approximated by the formula WR = 10 log $(1/r_u)$–1.5 (valid for $0.02 < r_u < 0, 7$) (Romana, 2003).

Table 1 shows a good general agreement of this formula with the values proposed by Bieniawski (1989).

Around the dam r_u changes in every point depending on the valley geometry, the water level, and the efficiency of the grouting curtains (if exist). Therefore it would be necessary a three-dimensional flow model of the dam and the surrounding terrain to determine the r_u exact values. Anyway $r_u > 0.4$ for almost all the upstream

Table 1. Relationship between (WR) and r_u.

WR	15	10	7	4	0
r_u (Bieniawski)	0	0–0.1	0.1–0.2	0.2–0.5	>0.5
r_u (Romana)	0.02	0.07	0.14	0.28	0.7

points, so the WR parameter would get values of less than 2.5. The value of r_u downstream depends on the performance of grouting and drainage curtains. There are no criteria for dams in the Bieniawski formulations and many authors have used values of 7 or 4 for WR to be in the safe side. Exact consideration of WR is a secondary matter, as normal operator error in RMR geomechanics classification is around 5. Anyway formula 1 would allow exact calculation of WR if needed/required.

Furthermore the compressive strength of the rock will diminish heavily when saturated (see Romana and Vasarhely, 2007) and its rating probably will halve. So a very crude way of taking account of the water effect on E_m would be to subtract around 15–20 points of the "dry" values of RMR. If we accept the Serafim-Pereira formula (see later) for determination of E_m from RMR E_m (GPa) = 10(RMR-10)/40 the value of E_m (dry) would be approximately three times the value of E_m (saturated), for 10 < RMR < 70. This result is not consistent with published data, which allow for a reduction on the order of 40–50% for E_m when saturated. Therefore a rule of thumb could be to subtract 10 points of RMR (dry) to obtain E_m (water saturated). It is interesting to note that this is congruent with prior versions of RMR (before the 1989 version which actually has become the "standard" one), and it is also the preferred method in Hoek's GSI index practice.

Anyway it seems that the water consideration has been in the past a serious handicap not only for the accurate determination of E_m by correlations with RMR, but also for the use itself of RMR in dams. The problem could be avoided with the Hoek's definition of basic dry RMR (see above) for calculations.

4 GUIDELINES TO CHECK THE STABILITY OF DAMS AGAINST SLIDING

Bieniawski and Orr (1976) proposed the a set of correction factors for the effect of the joints orientation in horizontal stability of gravity dams (Table 2) "based on experience and on considerations of stress distributions in foundation rock masses as well as on an assumption that in a dam structure both the arch and the gravity effects are present" (sic).

Snell and Knigth (1991) approached systematically the problem of the dam stability taking account of all the forces and stresses acting on the dams. Based in their study and others it appears that a different set of adjusting factors must be applied. Table 3 shows these new (modified in 2010) tentative correction factors according the main discontinuities orientation. The numerical rating values proposed originally by Bieniawski have been retained.

Table 2. Adjusting factor for gravity dams stability after joints orientation (Bieniawski & Orr, 1976).

	VF	F	Fa	U	VU
Type of dam gravity	Very favorable	Favorable	Fair	Unfavorable	Very unfavorable
Dip (°)	0–10	30–60	10–30 DS	10–30 US	60–90
Rating	0	–2	–7	–15	–25

(°) DS dip downstream/US dip upstream.

Table 3. Adjusting factors for the dam stability R_{STA}, according joints orientation (modified in 2010 from Romana, 2003a).

	VF	F	FA	U	VU
Type of dam	Very favorable	Favorable	Fair	Unfavorable	Very unfavorable
Fill	Others	10–30 DS	0–10 A	–	–
Gravity(*)	10–60 DS	30–60 US	10–30 US 60–90 A	0–10 A	–
Arch	30–60 DS	10–30 DS	30–60 US	10–30 US 60–90 A	0–10 A
R_{STA}	0	–2	–7	–15	–25

DS dip downstream/US dip upstream/A any dip
(*): CVC (Conventional Vibrated Concrete) RCC (Roller Compacted Concrete) hard fill.

The value of DMR_{STA} (related to the dam stability against sliding) is:

$DMR_{STA} = RMR + R_{STA}$ (modified in 2010 version) where R_{STA} is the correction factor for dam stability (Table 3).

When the dip direction of the governing joint is not almost parallel to the downstream-upstream axis of the dam the danger of sliding diminishes due to the geometrical difficulties to slide. The correction factors above mentioned would be valid for the cases where the angle between the dip direction of the governing joints and the axis of the dam along the river is less than 30º. In other cases a more detailed study must be done.

Actually there are no data which allow establishing a correlation between the value of DMR_{STA} and the degree of safety of the dam against sliding. As a rule of thumb we can suggest the following values as indications for the designer:

$DMR_{STA} > 60$	No primary concern
$60 > DMR_{STA} > 45$	Light concern
$45 > DMR_{STA} > 30$	Concern
$30 > DMR_{STA}$	Serious concern

These cannot be taken at all as numerical statements, but only as danger signals for the designer. Dam stability must, always and in any case, be checked by the designer taking account of the probable distribution of water pore pressure across the dam foundation and of the mobilized shear strength of the governing joints. About this question the Spanish National Committee on Large Dams states that "The study of the dam safety against sliding requires knowledge of the strength of the rock mass. The simple correlations between geomechanics classifications and rock mass strength is not well established for dam foundations" (SCOLD, 1999). Election of shear strength parameters must be done with sound rock mechanics criteria in relationship with laboratory test supplemented in some cases by field tests. This point is out of the scope of any geomechanics classification and the values proposed by Bienawski (based on RMR classes) are only rough estimations, and should not be taken account of for calculation.

5 GUIDELINES FOR EXCAVATION AND CONSOLIDATION GROUTING OF DAM FOUNDATIONS

The most usual requirement for the quality of the rock foundation for a concrete dam was something as "good quality, sound rock, and fresh, not weathered". Sharma (1998) is more specific demanding that "the entire (foundation) area should be stripped to firm rock capable of withstanding the loads. Any layer of weak or soft material has to be excavated and replaced with concrete". He prescribes dental concrete treatment filling with concrete any open (or filled with soft fill) joint according USBR rules.

In most cases the foundation is excavated until class II rock in the central part of the valley (where the dam is higher) and until class II–III rock in the abutments. Spillways are founded, if possible, in class I rock. The most frequent parameter to be quantified in order to check the rock quality was the celerity of P-waves, measured by geophysical methods. Actually most rock masses are investigated and described with the ISRM suggested methods, which allow for a quick classification work.

It is desirable to gather data on the RMR value of dam foundations and this author is working in that. Actually some simple guidelines can be tentatively proposed (Table 4) for the depth of foundation excavation and for the required consolidation grouting of some few meters deep below the surface of foundation excavation.

Table 4. Guidelines for dam foundation excavation and consolidation grouting (Romana, 2003a).

Dam type	Excavate to RMR_{BD} (+)	Consolidation grouting according RMR_{BD}		
		Systematic	Spot	None
Earth	–	–	?	–
Rockfill	>20 (>30)	20–30	30–50	>50
Gravity	>40 (>60)	40–50	50–60	>60
Arch	>50 (>70)	50–60	60–70	>70

(+) minimum (desirable) -gravity dams include CVC, RCC and hardfill concrete -rockfill dams included are the ones with concrete –CFRD- or asphaltic –AFRD-face.

6 CORRELATIONS BETWEEN E_M AND RMR

6.1 *General*

E_m modulus can have different values depending on the direction of the principal stress. In stratified rock mass, and/or with a governing joint orientation, the equivalent elastic modulus of deformation is the weighted arithmetic mean of the deformation modulus of the strata (when the stress is parallel to them), and the weighted harmonic mean (when the stress is perpendicular to them).

Therefore the perpendicular deformation modulus is always the minimal one and the parallel deformation modulus is always the maximal one. The difference between both of them gets wider as anisotropy of the rock mass increases. Barton (1983) proposed the following formulae:

$$E_{min} = 0.4\ E_{mean},\ E_{max} = 1.6\ E_{mean}$$
$$\text{and therefore } (E_{max} - E_{min}) / E_{mean} = 1.2.$$

This would imply a relationship of 4 between maximal and minimal values of deformation modulus, which, according to Barton, is confirmed by other data published by Rocha (1964) and Bieniawski (1978), and is possibly adequate for rock masses very anisotropic and/or much bedded with beds of very different nature. In more homogeneous anisotropic rock masses, however, the relationship between maximal and minimal values of the deformation modulus seems much smaller, in the order of 1,4. Many authors have published data gathered from in situ tests. Some ones from important projects are resumed in Table 5.

Water also reduces both the strength and the equivalent deformation modulus when the rock mass is saturated, a very important effect in dam foundations. The first papers neglected this water effect. The introduction of GSI (by Hoek) included a very simple way to consider the water effect: to evaluate

Table 5. Some data from in situ tests on the relationship E_{max}/E_{min}.

Site	Rock mass	E_{max}/E_{min}	Reference
Contra dam	Gneiss	2,5	Lombardi (1967)
Tamzaourt dam	Sandstone Siltstone	1,3 1,9	Jaoui et al. (1982)
Ridracoli dam	Marl Sandstone	1,3 1,4	Oberti et al. (1986)
Alcantara dam	Decompressed slate/schist Decompressed slate/schist after consolidation grouting Slate/schist after consolidation grouting	1,5 1,2 1,14	Navalón et al. (1991)
Colbun power plant	Andesite	1,4	Van Sint (1993)
Rules dam	Micaceous schist	3,3 (4 at noval-charge)	Nevot et al. (1993)

RMR as if the rock mass were fully dry (with a value for the water parameter WR of 15) and to introduce water pore pressure in computations (see Bieniawski 2000). But that implies that neither the strength, nor the deformation modulus of the rock mass changes, and this hypothesis doesn't take account of the reduction both of the strength and of the deformability of the rock mass when saturated.

Figure 1 (Pells, 1993) shows a Deere-Miller diagram containing data from compression tests (failure strength and deformation modulus at 50% of failure strength) in dry and saturated Hawkesbury sandstone. Saturation implies a reduction almost proportional in both parameters, but the relationship between them would remain approximately constant.

6.2 *Proposed correlations between E_m and RMR*

Bieniawski (1978) proposed the correlation

$$E_m\ (GPa) = 2\ RMR - 100$$

and it can be seen (Fig. 2a) that this formula may be adequate for RMR > 65, has a wide scattering for 55 < RMR < 65, (a very common interval in practice) and cannot be used for RMR < 55.

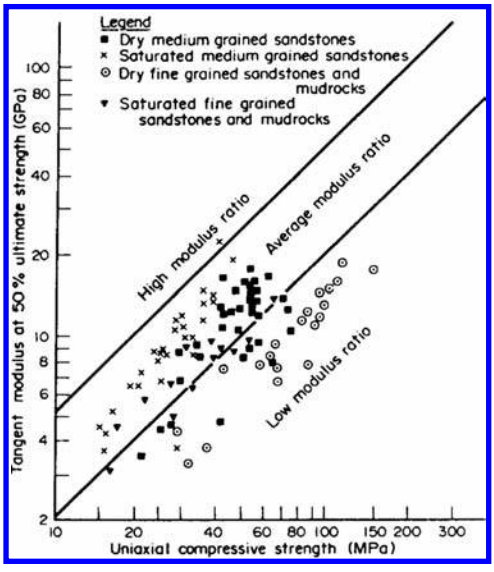

Figure 1. Strength and modulus data on Hawkesbury Sandstone (Pells, 1993).

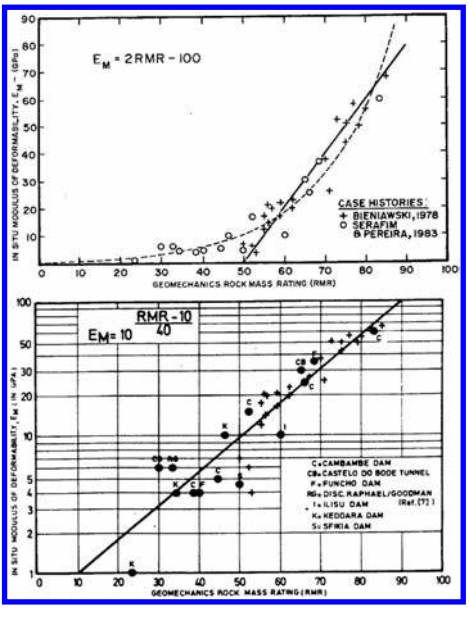

Figure 2. Correlations between the in-situ modulus of deformation and RMR according to: (a) Bieniawski (1978); (b) Serafim & Pereira (1983).

Bieniawski recommends "good engineering judgment" when using his formula.

Bieniawski correlation has been modified by Serafim & Pereira (1983) for all values of RMR

$$E_m\ (GPa) = 10^{(RMR-10)/40}$$ (Fig. 2a, 2b)

a formula that has been widely accepted in dam practice. This formula works better for RMR > 34. There were only three cases with lower RMR (22, 30 and 33) in their data bank, and the correlation was worse in all three of them. Figure 2a shows Bieniawski and Serafim & Pereira correlations. Both are practically equivalent for RMR > 65, and both show a poor accuracy for smaller values of RMR.

In practice most of engineers follow procedures similar to the guidelines of USA Federal Energy Regulatory Commission (1999): "for RMR > 58 use Bieniawski formula; for RMR < 58 use Serafim-Pereira one". The RMR = 58 value appears to have been selected because it is the abscissa of the intersection between both curves.

Hoek and Brown (1997) proposed a modification of Serafim & Pereira formula in order to take account of the influence of the intact rock compressive strength σ_c (MPa):

$$E_m\ (GPa) = (\sigma_c/100)^{1/2} \times 10^{(GSI\ -10)/40}$$

This formula is proposed for all values of RMR. The Serafim & Pereira original value would correspond to rocks of 100 MPa compressive strength, which is not at all usual in the rock masses studied by Serafim & Pereira, with RMR < 50. This modification by Hoek & Brown could be useful for high values of RMR and allows for the water effect testing dry and saturated probes. Hoek notes that this "equation appears to work reasonably well in those cases where it has been applied. However, as more field evidence is gathered, it could be necessary to modify this relationship". This formula is being used in practice but there is no factual data supporting its use in dams practice for rock masses of medium or low hardness.

In a paper for the 2002 NARMS, Hoek, Carranza-Torres and Corkum have introduced an additional "disturbance factor" D

$$E_m(GPa) = (1\text{-}D/2) \times (\sigma_c/100)^{½} \times 10^{(GSI-10)/40}$$

The new factor D "allows for the effects of blast damage and stress relaxation", and can be estimated according to guidelines given by Hoek for tunnels, slopes, and pit/quarries, but not for dams. As excavations for dams' foundations are, as a rule, very careful, D should be very low, but it cannot be 0 because of the rock mass decompression when excavating the dam foundation. Tentative guidelines for D value near the dam foundation are as follows (Romana 2003a):

–Good rock mass, normal blasting D = 0,4
–Any rock mass, controlled blasting D = 0,2
–Poor rock mass, mechanical excavation D = 0,2

There is a new formulation (deduced from a Chinese database) by Hoek and Diederich.

6.3 *Recommended formulae*

Hoek's formulas are widely used for any rock structure. But with the published data on dams' projects there is only actual basis, in the opinion of this writer, for the following recommendations:

$RMR_{BD} > 60$	Use Bieniawski formula
$60 > RMR_{BD} > 35$	Use Serafim & Pereira original formula.
$35 > RMR_{BD}$	Use as, a guess, Serafim & Pereira formula

RMR_{BD} is included because all the deformation modulus testing, in the field and/or the laboratory has been done in dry conditions. But the values in saturated conditions could be minor. RMR_{BD} could be substituted by RMR_B but the results are not very different.

There are no data supporting normalized Hoek variations of Serafim & Pereira formula. The results obtained when applied to good-medium quality rock masses (e.g. RMR class II–III) with medium hard rocks (e.g. σ_c = 50 MPa) are not consistent with the published data on E_m.

6.4 *Effect of anisotropy on E_m*

E_m depends on the anisotropy of the rock mass and the maximum stress direction. It is possible to take account of this effect with the following rules of thumb:

–In very anisotropic rock masses
$E_{max}/E_{min} = 4$.
Add 8 to RMR_B to obtain E_{max} and subtract 16 to RMR_B to obtain E_{min}.

–In slightly anisotropic rock masses
$E_{max}/E_{min} \sim 1.4$.
Add 2 to RMR_B to obtain E_{max} and subtract 1.4 to obtain E_{min}.

Such values are only approximate but can be useful when making sensitivity analysis of stress fields in dam foundation to get something similar to extreme situations. The best solution would be to apply some correction factor depending on the orientation of the joint(s) which define the more competent "strata" in the rock mass.

7 INFLUENCE OF THE FOUNDATION DEFORMABILITY ON THE DAM BEHAVIOR

7.1 *General*

There is a general agreement between dam engineers on the fact that two cases are dangerous for the normal behavior of a concrete dam: if E_m varies

Table 6. Effect of Ec/Em on arch dam behavior (Rocha, 1964).

Ec/Em	Influence on dam	Problems
<1	Negligible	None
1–4	Low importance	None
4–8	Important	Some
8–16	Very important	Serious
>16	Special measures	Very dangerous

Table 7. Effect of Ec/Em on arch dam behavior (Rocha, 1975, 1976).

Ec/Em	Influence on dam	Problems
<4	Negligible	None
4–8	Low importance	Minor
8–16	Important	Some
>16	Very important	Moderate to big

widely across dam foundation, or if Ec/Em reaches certain values (Ec being the deformation modulus of concrete).

Rocha (1964) established the most followed rule for arch dams (Table 6) in a paper which has become a "classic" reference for dam designers.

Ec/Em < 4 allows for an easy behavior (and "high cost tests in the foundation exploration could be dispensed with" according Oliveira, 1990). The minimal sure (but with problems) value of E_m for an arch dam would be around 5 GPa. The reported cases of arch dams founded in rock masses with E_m < 5 GPa show serious problems (cracking included) because of the low value of E_m.

Rocha et al. (1974) presented data on the Alto Rabagao dam "built in a very deformable foundation", with a modulus relationship Ec/Em of 20 above elevation 830 (maximum water level 880). "Tensile stresses (were displayed) … in some points near the foundation downstream in the left bank. (As) these tensile stresses (were) specially relevant … it was recommended … reinforcement parallel to the downstream face and … to the ground".

Silveira et al. (1991) in a paper with the title "Influence of foundation heterogeneity on safety of arch dams" which was presented at 17th ICOLD Congress in Vienna analyzed the stresses in several built arch dams with very different values of Ec/Em both in the after-construction state and the predicable aged state after many years. Their conclusions were that "this influence (of the heterogeneity of foundation deformability) on the behavior of arch dams for ordinary scenarios is (well) summarized (in Rocha table) and only for large heterogeneities this influence is remarkable for arch dams", "the heterogeneity of foundation decreases the safety factor and thus the capability of the dam to resist to ageing. However … this reduction is only remarkable for larger heterogeneities".

In later papers Rocha (1975, 1976 in Portuguese) extended his work to gravity dams (Table 7).

Some gravity dams where Ec/Em = 20 have needed extraordinary measures and very slow first impounding (several years) to consolidate the foundation. The existence of joints in concrete dams helps to cope with relative deformability problems. This may be the main reason in the changes in the design of RCC concrete dams, from the first dams with almost no joints to the actual standards with more frequent joints. Nevertheless RCC concrete gravity dams are less prone to problems than CVC concrete dams due to the lesser value of Ec.

7.2 *Value of Ec in CVC and RCC concrete*

Andriolo (1995) gave a detailed report comparing properties of CVC and RCC concrete. Fig. 3 shows the data on Ec from 5 CVC dams and 13 RCC dams (ost of them brazilian). The mean data gives relationships of:

$E_{RCC} = 0.40$ ECVC (at 7–28 days)
$E_{RCC} = 0.55$ ECVC (at 90 days)

At 90 days ECVC varies between 28 GPa and 51 GPa with a mean value of 39 GPa, whereas ERCC varies between 11 GPa and 32 GPa with a mean value of 22 GPa. There is a great variation, depending on the cementitious material content, but we can assume the following mean values for each class of concrete:

CVC ECVC = 30/36 GPa,
RCC ERCC = 20 GPa (or less in many cases).

7.3 *Value of E_{hard} in hardfill dams*

There are only very few hardfill dams already built. However ICOLD, in his Bulletin on the State of Art of RCC dams (2000), proposes $E_{hard} = 10$ GPa, (or less), for a mixture with compressive strength of 9 MPa, values which are congruent with the data from the Lower Monción Dam, built with hardfill and already in operation (Capote et al, 2003), and will be used in DMR method until more data could be gathered.

The same Ec value can be adopted for the revision of old dams, built with poor concrete of low strength, and / or deteriorated by ageing.

7.4 *Guidelines for danger of abnormal behavior due to the difference on deformabilities between dam and foundation*

Zeballos and Soriano (1993) have published the results of Zeballos Ph. D. thesis (Polytechnic

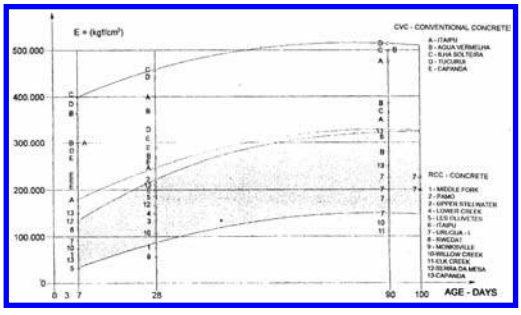

Figure 3. Modulus of elasticity values from RCC and CVC concrete in dams (Andriolo, 1995).

Table 8. Deformability problems in concrete dams according value of DMR_{DEF} (modified from Romana 2003).

Dam Ec (GPa)	Height (m)	Normal	Problems	Serious problems
Arch	<100	>50	40–50	<40
36 GPa	100–150	>65	50–65	<50
	150–200	>75	60–75	<60
Gravity	<50	>40	25–40	<25
CVC	50–100	>50	40–50	<40
30 GPa	100–150	>60	50–60	<50
Gravity	<50	>35	20–35	<20
RCC	50–100	>45	35–45	<35
20 GPa	>100	>55	45–55	<45
Hardfill	<50	>30	15–30	<15
10 GPa	50–100	>40	30–40	<30

University of Madrid), an extensive and intensive study on the effects of Ec/Em value on gravity and arch dams.

Table 8 (gathered using their data and others) shows the different ranges of RMR_{DEF} related to the different ranges of possible problems in the dam due to the differences of deformability between the dam and his foundation.

DMR_{DEF} (RMR related to deformability by the Serafim & Pereira formula) depends on E_m (when the rock mass is saturated) and can be estimated with WR = 5 (a mean value of the correction factor which correspond to a mean value of $r_u = 0.25$).

8 USE OF DMR IN THE APPRAISAL OF SAFETY CONDITIONS IN OLD DAMS

At this moment a big number of old dams are in operation. Many were built before rock mechanics beginning (during the 1950–1960 decade) and were designed based only in the good engineering sense and experience of the designers (engineers and geologists). The existing information on the foundation conditions is very scarce and never is presented following the actual "good practice", which has been developed in the last 30 years.

The analysis of the safety conditions of these old dams has become mandatory in all countries. The DMR geomechanics classification is a good method to gather orderly sound geomechanics information about the rock mass foundation and allows for a preliminary appraisal of the "weak points". DMR has been used in Spain to tis purpose by different workers (and the author himself), as a first tool, in the safety analysis of old (more than 50 years) gravity and masonry dams with good results. All of them are in operation and show no visible signs of serious damage. The concrete quality varies from good to very poor (i.e. Balagueras dam with σ_c~10 MPa.

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DEFINITIONS

WR	Rating of the fifth parameter (water) in RMR
RMR_B	Basic RMR, with no adjusting factor for joints orientation.
RMR_{BD}	Dry basic RMR, with no adjusting factor for joints orientation. WR = 15
R_{STA}	Adjusting factor for joint orientation (dip) related to dam stability (Table 3)
CF	Geometric correcting factor for relative orientation of joints and dam axis. $CF = (1-\sin\lvert\alpha_d-\alpha_j\rvert)^2$

α_d Direction upstream-downstream of the dam axis
α_j Dip direction of the significative governing joint for dam stability.
DMR_{STA} DMR related to dam stability.
$DMR_{STA} = RMR_{BD} + R_{STA}$
E_m Deformation modulus of rock mass.
E_{max} Maximal deformation modulus of rock mass.
E_{min} Minimal deformation modulus of rock mass.
E_c Deformation modulus of concrete
E_{CVC} Deformation modulus of conventional vibrated concrete
E_{RCC} Deformation modulus of roller compacted concrete
E_{hard} Deformation modulus of hardfill

DMR_{DEF} RMR related to relative deformability, with WR = 5, and no adjusting factor for joints orientation. (Tab. 8)

APPENDIX: GUIDELINES FOR THE ESTIMATION OF DMR AROUND EXISTING DAMS

Site election

DMR requires the field estimation of RMR. Sites for estimation must be selected with care. Obviously rock outcrops near the lower part upstream of the dam must be the first option. But outcrops nearby the dam foundation may have been affected by the blasting for the foundation excavation.

Outcrops can be irregular in size and form and may not allow an accurate estimation of all parameters included in RMR, especially joints spacing in every joints family. It is very convenient to check the results in the slopes along the roads to the dam.

If possible a ditch must be excavated to sound rock near the dam in order to check alteration patterns with depth.

Compressive strength estimation

The compressive strength of the intact rock can be assessed in several approximate ways. Normally the compressive strength will be lower near the surface. Prudent allowances must be done for this fact.

Usually exact values of unconfined compressive strength (UCS) are not needed for classification purposes. So, frequently, indirect estimations are done in the field, to be checked afterwards against laboratory tests results.

Contrary to Bieniawski, and many others, indications, there is no actual a fixed quotient value between compressive strength and Point-Load Index (PLI), valid for all classes of rock. If results of point-load test are to be used the correlation with compressive strength must be established with actual laboratory test in each class of rock. Approximate quotient between compressive strength (UCS) and Point-Load Index (PLI) can be taken from Romana (1998) for different rock classes:

ROCKS	UCS/PLI
• Igneous/ultrabasic	20–25
• Quartzite	18–24
• Gneiss	16–24
• Metamorphic (strong)	(15)–22
• Metamorphic (medium)	10–(17)
• Lutite, limolite, greywacke	10–15
• Well indurate limestone	(18)–24
• Poorly indurate limestone	12–(18)
• Very porous sedimentary	5–10

(figures in brackets are estimations)

The relationship between dry (UCS_{dry}) and saturated (UCS_{sat}) unconfined compressive strength can be estimated roughly from Romana and Vasarhely (2007).

ROCKS	UCS_{sat}/UCS_{dry}
• Well indurate strong	0,80–0,90
• Cemented medium strength	060–0,70
• Soft argillaceous	~ 0,30

A correction is necessary because rock around (and below) the dam foundation will be saturated. So laboratory test (done on dry or ambient humidity probes) will over-estimate the unconfined compressive strength.

RQD

Most of the exploration borings will be vertical ones and can mislead on the value of RQD in other directions.

In other cases many borings will be more frequent in areas of special geological complexity (or around faults), in order to clear the geological general picture and the details. In these borings RQD results will be minored.

Probably the dam foundation will be deep enough (it is necessary to check this) to avoid the decompression joints parallel to the terrain surface. In this case decompression joints must not be taken account of.

The Palstrom formula gives a good estimation of the "global" RQD of the rock mass and must be used even if there are borings (checking the results for compatibility).

$RQD = 115–3.4\ Jv\ (RQD \leq 100)$
$Jv = \Sigma\ 1/si$ (extended to all joints' families)

where Jv is the number of joints in a cubic meter and si is the spacing (in meters) in the family i of joints.

Spacing and frequency of joints

Very often joints are more frequent at the surface that inside the rock mass. This is due to different causes (weathering, solution, others) and the effect is difficult to quantify.

In man made slopes more joints will exist (and be open) due to blasting

State of joints

The same surface effect makes the joints be more open at surface. Weathering can be also more important at surface. Joints will be more closed inside the rock mass.

Water parameter

WR should be estimated from ru values below the foundation and around the dam. These values are difficult to guess if there are not enough piezometers below the dam or if some of them malfunctions.

Nevertheless data on possible water flow across joints is very important and must be gathered in the field work. Bieniawski descriptions can be used for the joints watering state (flowing, dripping, wet, humid, dry).

It is important to make allowance for possible changes in groundwater levels due to an actual lower reservoir level and/or the change to summer/dry weather conditions. Vegetation gives cues to that. Also humidity in the face of a slope can left a difference of colours when dry.