Enhancing Urban Environment by Environmental Upgrading and Restoration

Edited by

Jiri Marsalek, Daniel Sztruhar,
Mario Guilianelli and Ben Urbonas

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IV. Earth and Environmental Sciences – Vol. 43

Enhancing Urban Environment by Environmental Upgrading and Restoration

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Series IV: Earth and Environmental Sciences – Vol. 43

Enhancing Urban Environment by Environmental Upgrading and Restoration

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PREFACE

Management of urban waters, particularly with respect to their sustainability, remains to be a great challenge in all regions of the world. Indeed, as cities and urban centres continue to grow, urban waters are often degraded, encroached upon or even totally destroyed, but at the same time, the requirements of the urban population on such waters keep growing. Furthermore, the older water management infrastructure is becoming inadequate or obsolete, either because of insufficient capacity, or structural degradation, or inability to meet new requirements of sustainable development. Under these circumstances, there is a need to upgrade the existing facilities and restore the affected urban streams, lakes and wetlands. Such upgrading and restoration activities were the main subjects of the NATO Advanced Research Workshop on Enhancing Urban Environment: Environmental Upgrading of Municipal Pollution Control Facilities and Restoration of Urban Waters, which was held in Rome, Italy, from November 5 to 8, 2003.

After receiving the NATO support grant, the organisers have confirmed keynote speakers, selected workshop participants, finalised the workshop programme, and held the workshop at the Aula Magna of the Christian Brothers' Centre (Casa la Salle) in Rome. There were 52 full-time participants at the workshop, from 17 countries, and additional observers also audited the workshop program. Extensive experience of workshop participants in this field is reflected in the workshop proceeding, which comprise 33 selected papers. Additionally, a number of posters were also on display during the workshop. Finally, whenever trade, product or firm names are used in the proceeding, it is for identification and descriptive purposes only, without implying endorsement by the Editors, Authors or NATO.

The proceedings that follow reflect only the formal workshop presentations. Besides these papers, posters, and extensive discussions, there were many other ways of sharing and exchanging information among the participants, in the form of new or renewed collaborative links, professional networking and personal friendships. The peaceful atmosphere of the Casa la Salle Centre, and the excitement of meeting in the "eternal city", contributed to the success of this workshop. For this success, the editors and organisers are indebted to many who helped stage the workshop and produce its proceedings, as listed in the Acknowledgement.

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ACKNOWLEDGEMENT

This Advanced Research Workshop (ARW) resulted from hard work of many individuals and organisations. The workshop was proposed and directed by Dr. Jiri Marsalek, National Water Research Institute (NWRI), Environment Canada, Burlington, Canada, and Dr. Daniel Sztruhar, Department of Sanitary Engineering, Slovak Technical University, Bratislava, Slovakia. They were assisted by three other members of the workshop Organising Committee, Ben Urbonas, Urban Drainage and Flood Control District, Denver, CO, USA, and Drs. Mario Giulianelli and Dario Marani, Water Research Institute, Rome, Italy.

The ARW was sponsored by NATO, Public Diplomacy Division, Collaborative Programmes Section, in the form of a grant; by employers of the members of the organising committee, who provided additional resources required to prepare the workshop and its proceedings; and, by the Centre for Studies of Urban Drainage (CSUD, Centro Studi Idraulika Urbana) stationed at Milan Polytechnic, Milan, Italy. CSUD recruited a number of workshop speakers, promoted the event, and secured a sponsor for the workshop banquet, Steinhardt GmbH – Innovative Technology for Water Pollution Control, Taunusstein, Germany.

The early preparatory work was conducted by the Slovak Technical University team (led by Daniel Sztruhar, assisted by Ivana Mahrikova, Dana Barlokova and Dr. Stefan Stanko), and the NWRI team (led by Jiri Marsalek, assisted by Quintin Rochfort and Mike Donnelly). Local arrangements were carried out by Drs. J. Marsalek, Mario Giulianelli and Dario Marani, with Peter Marsalek providing liaison with the Casa la Salle Conference Centre. At the Casa la Salle Conference Centre, the provision of excellent services was managed by Messrs. Massimo Pallotta and Maurizio de Paolis.

Special thanks are due to Dr. A.H. Jubier, Programme Director, Environmental Security, NATO, who provided liaison between the workshop organisers and NATO, and personally assisted with many tasks.

Finally, the organisers are indebted to all the above contributors and, above all, to the participants, who made this workshop a memorable interactive learning experience for all involved.

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RETROFITTING URBAN DRAINAGE SYSTEMS USING BEST STORMWATER MANAGEMENT PRACTICES – SOME SCANDINAVIAN EXPERIENCES

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1. Introduction

Development of alternative stormwater management practices which would be more sustainable than the traditional ones has come into focus in the Scandinavian countries during the 1990s. Attention is paid to mitigating the effects of wet weather discharges from separate storm sewers and combined sewer overflows, because these effects are now considered more critical after discharges from wastewater treatment plants have been controlled to acceptable levels. Methods that are cost efficient and make stormwater visible are promoted. Concern is also given to treatment facilities on different spatial scales, and to operation and maintenance practices.

This paper presents recent experiences in Denmark and Sweden related to the use of stormwater 'best management practices' (BMPs). First, the concept of the present sewer systems in both countries is explained and some future developments are outlined. Hereafter, selected recent case studies are presented and finally, the findings and other issues of current interest in Denmark and Sweden are discussed.

2. Structure of urban runoff systems in Sweden and Denmark

2.1 EXISTING SEWER SYSTEMS

The concept of drainage systems serving urban areas was conceived around 1850 in Sweden and Denmark when cholera epidemics raged European cities. Many old town centres are still today served by combined sewer systems (Figure 1, top left) but most urban areas built during the 1950-70s building boom and hereafter are served by separate sewer systems (Figure 1, top right). The idea of separate sewer systems was adopted earlier in Sweden than in Denmark, and separation of old combined systems has been systematically carried on for many years in Sweden. This is the reason that today, the fraction of urban areas in Sweden that are still served by combined sewers is only 15% (based on pipe length), compared to 50% in Denmark.

In Denmark, combined and separate systems often co-exist within the same urban

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areas, and separate wastewater and stormwater systems are sometimes connected to existing combined sewers situated downstream from new developments. However, in most cases, stormwater runoff is intentionally discharged untreated into receiving waters. For both combined and separate systems, plants have been constructed to treat the wastewater since the 1960s and upgraded to comply with the present discharge limits during the 1980-90s. However, combined sewer overflows (CSOs) and stormwater outfalls still remain, and they become more visible and receive more attention in Denmark and Sweden now that the wastewater pollution discharges have been significantly reduced.

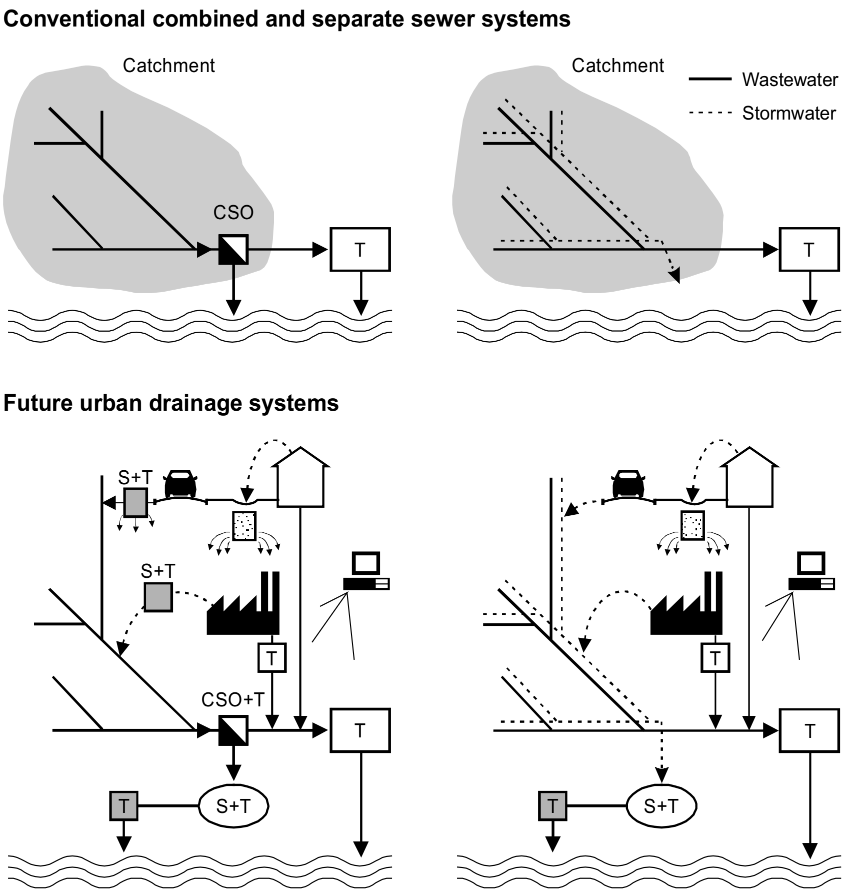

Figure 1. Layout of existing and future urban drainage systems (S ~ storage and equalisation of stormwater flows, T ~ treatment).

2.2 FUTURE SEWER SYSTEMS

Figure 1 (bottom right) illustrates what separate sewer systems may well develop into. Importantly, all stormwater discharges will undergo some kind of equalisation and treatment using storage facilities (e.g. a tank, pond or wetland) in combination with a treatment process for polishing the effluent. Sometimes storage and treatment will be combined in vegetated wetlands that may also function as public amenities. Roof runoff will preferably be infiltrated locally, leaving the stormwater sewers to convey only the heavily polluted runoff from streets and roads and industrial areas.

For combined systems, a similar development can be foreseen (Figure 1, bottom left). Many CSO structures will be abolished and the remaining ones will be fitted with large storage facilities and various treatment devices to prevent gross pollutants and other constituents from impacting the receiving waters. Roof runoff will be infiltrated locally but since flooding of basements and transport routes is unacceptable, excess amounts of stormwater must be allowed to escape to the combined sewers. Runoff from streets and roads is partly infiltrated and partly discharged to the combined sewers after equalisation; however local treatment or street sweeping will be mandatory to reduce traffic related pollutants from entering into the combined sewer and ending up in sewage sludge.

Undoubtedly, future sewer systems will be more complex than the conventional ones, as structural BMPs for control of stormwater runoff will be applied on the local scale (e.g., infiltration), the sewer catchment scale (e.g., ponds and treatment) and the receiving water scale (e.g., retrofitting of river stretches), and as needs will also be met by non-structural BMPs such as ban of products, control of building materials, and street sweeping.

3. Recent Experience in Denmark and Sweden

3.1 CHARACTERISTICS OF STORMWATER RUNOFF

Focus with respect to stormwater quality has mostly been on selected key-constituents such as suspended solids, chloride (from road salting), organic biodegradable matter (BOD/COD), E-coli, heavy metals and polyaromatic hydrocarbons (PAH). Bio-accumulative compounds have received the most attention because they end up in parts of the environment directly exposed to humans (i.e., sediments). Pesticides and other organic chemicals related with use of products and erosion of the built environment have only recently come into focus.

The ‘traditional’ list of key-substances is however not exhaustive. Society produces thousands of new chemicals every year, and most of them will find their way into stormwater runoff and eventually be identified as the analytical detection limits are continuously improved. The international literature reports on an increasing number of investigations that focus on chemicals in stormwater runoff. Approximately 600 individual constituents have already been identified as potentially present in stormwater runoff, and efforts are therefore made to define procedures for identifying the most relevant ones for further studies within a certain context [1].

In both Denmark and Sweden, several investigations during the 1980-90s have studied the chemical composition of stormwater runoff from traffic areas, mostly focusing on selected heavy metals (Cd, Pb, Cu, and Zn) and PAHs. Currently, national surveillance programs require that wet weather discharges from combined systems be quantified annually, either by measurements or calculations. In Denmark, discharges from separate systems are also included in the surveillance program, and several monitoring stations have been set up to improve the basis for estimating pollutant loads at un-gauged locations. In addition to N, P and COD that have been monitored for many years [2], more than 60 individual 'environmental contaminants' are included in the new monitoring program [3], and as an alternative approach, some investigations recently studied the eco-toxicity of stormwater as a supplement to identifying and quantifying individual compounds [4, 5]. The outcome of such new initiatives is not complete yet but the initiatives illustrate the present focus on 'new' chemical pollutants and eco-toxicity.

3.2 BMPS IN SEPARATE STORMWATER SYSTEMS

3.2.1 Oil separators

Oil separators are often used to improve the quality of stormwater from industrial areas, and sometimes from traffic areas in cities. Different kinds of oil separators are used, most often involving sedimentation and flotation processes.

A lamella separator was installed at the Blommensberg area in central Stockholm 1995, treating stormwater runoff from a road with a traffic density of about 120,000 vehicles/day. Flow proportional samples of inflows and outflows were collected during 32 rain events [6]. The average results showed that of the 900 kg of suspended solids entering the separator, approximately 160 kg were retained, which corresponds to 17% removal. The removal of oil was 11%. This figure is uncertain because of low concentrations in incoming water. The removals of heavy metals varied between 9 and 14%. The relatively low removal efficiencies were due to a short detention time in the separator (about 8 minutes), and the necessity of by-passing or placing a detention tank upstream of the separator was stressed.

In Denmark, similar investigations have been made, but it is important to stress that the relatively low removal efficiencies are reported for ‘passive’ periods when no maintenance is conducted. To achieve reasonable removal efficiency it is important that the grit and floatables chambers are regularly emptied to prevent the accumulated material being flushed out during the next heavy rain storm [7].

3.2.2 Open ponds

Open ponds are generally used in both Denmark and Sweden to reduce peak flows and retain pollution from separate systems, in particular highways, but there is generally little knowledge about the performance of such ponds. Recently, Chalmers University of Technology undertook a 2-year, in depth, study of two ponds [8, 9]. Both ponds were equipped with automatic flow-proportional samplers.

It was concluded that ponds can be an effective pollutant control measure and that the removal efficiency increases with the pond specific area up to about 250 m^3/reduced ha (2.5 mm). It was also concluded that pond sediments are severely polluted, can be used for characterising pollutant loads from urban catchments and that they should be disposed in a proper way [10]. Thus stormwater ponds should be regarded as treatment facilities and not as habitat for wildlife. It was also concluded that heavy metal concentrations in the pond effluent were critically high compared to

Swedish guidelines for lakes and streams [10]. A literature study aimed at identifying future stormwater treatment systems for a re-developing part of Copenhagen arrived at a similar conclusion [11], thus emphasising the need for secondary treatment after passage through pond systems.

3.2.3 Vegetated wetland systems

Vegetated systems are sometimes used for equalisation and treatment of stormwater discharges, partly because of reduced costs compared with pond systems and partly because the public generally likes ‘green’ solutions that can be incorporated into a natural landscape and also serve as public amenity areas. So far, most experience with such solutions comes from Sweden.

One system consisting of an oil separator, sedimentation basins, a grassed filter strip and ponds with an average retention time of six days was constructed in Flemmingsberg/Huddinge during 1994-95 [12]. Despite some problems with the sampling (during 1995-98) the results showed that lead and zinc were reduced, while copper, mercury, chromium, suspended solids, COD, and TOC increased and BOD was quite constant. Interestingly, the wetland had a positive effect on the wildlife. Both in, and around, the wetland 10 to 12 different bird types have been seen, and also fish has been observed in the system.

Another stormwater management system located between Huddinge and Stockholm was completed in 1992 [13, 14]. The contributing areas are a city centre, roads and a sport arena, and the system includes a 1,000 m^2 grassed filter strip and a 10,000 m^3 banked basin in the lake Magelungen. Sampling during 1993-95 showed that the discharge of phosphorus from the grassed filter strip varied, depending on season, ranging from washout (during the summer) to a reduction of 70-80%. Also, the removal of nitrogen varied broadly, and the dissolved fraction was generally less reduced than the particulate-bound. It was concluded that the pollutants were sometimes washed out, and the grassed filter strip was hence taken out of use. The problems could be explained by difficulties to spread the water equally over the surface, and that the water took it own ways and cut channels in the surface. The results for the pond showed that the main reduction was due to sedimentation [13], i.e., particles and thus the associated phosphorus and metals were removed.

3.2.4 Constructed filter systems

It appears that post-treatment of effluents from stormwater ponds or wetlands are needed in some cases to achieve the limit values set for receiving waters. Removal of heavy metals in laboratory-scale constructed filters was studied at Mälardalen University. Column experiments were carried out with three different filter media: calcium silicate rock, zeolite and pine bark [15, 16], and different metal solution were used to simulate the runoff composition. The results showed high removal efficiency for low flow rates, however, as the flow rate increased the removal efficiency decreased. Concerning the removal efficiency and clogging aspects calcium silicate rock and zeolite were the best materials. The removal capacity varied between 0.6 kg/m^3 to 1.8 kg/m^3 depending on metals, filter substrates, and the volume of filtrated metal solution.

A system consisting of a detention pond, constructed filters and a constructed wetland has been tested in-situ in Sweden [17]. The results from the filter system showed that the calcium silicate rock clogged during the winter, and that water was not able to pass through the filter walls. The clogging was probably caused by cementation of the material.

Column and batch experiments using iron oxide coated sand (IOCS) as filter media have been conducted in Denmark [18]. The preliminary results show that significant removal of cationic heavy metals can be achieved even at very low concentrations, but that complexation with chloride (due application of road salt) and dissolved organic matter (due to e.g., algae growth during summer) may inhibit sorption processes, and that efficient back-washing of the filter media may be difficult without dissolving the IOCS. Filters based on IOCS will soon be installed in a pilot-scale plant in Copenhagen.

3.3 BMPS IN COMBINED SYSTEMS

3.3.1 Overflow-regulating modules

A newly developed system for combined reduction of water volume and pollution load from CSOs has recently been installed in Hørsholm [19]. The system consists of a pipe section in the drainage system with a relatively large diameter. The inflow is kept constant by a weir, thus allowing a relatively calm flow though the pipe section that enhances sedimentation. The outflow is separated into two parts; the underflow containing most particles is conveyed to the treatment plant and the cleaner surface flow is discharged to a nearby stream, which is intensively monitored. The volume of the pipe section is so large that it alone significantly reduces the yearly amount of overflow. However, experience shows that this system is 30-40% cheaper than traditional concrete basins with the same reduction of CSO pollution discharges.

The effect of the system is significant, with a pollution reduction of about 90%, although the reduction of the annual overflow volume is only 65%. These numbers are partly estimated on the basis of measurements and simulations. Further improvements are expected in the future by continuously regulating the flow to the treatment plant, based on information about its state. These conclusions suggest that the efficiency of detention ponds in separate storm systems may be improved by enhancing their settling conditions. For ponds in existing urban areas the efficiency may possibly be further improved by pumping the settled sludge to the treatment plant via the sanitary sewer.

3.3.2 Compact physical-chemical treatment

Efficient pollution removal for transient events can be obtained by using physical-chemical treatment. The Actiflo® treatment plant (Figure 2) was originally developed for drinking water treatment but a test period showed high removal efficiency for overflow water in combined sewer systems (SS: 80-98%, COD: 65-90%, N: 20-50%, P: 70-95% [20]). The outflow concentration does not exceed 0.05 mg/l total P even during variable load conditions. This high efficiency is due to injection of micro-sand and coagulants that enhances sedimentation in a lamella-settler. In this manner, a very compact but still efficient treatment system is possible.

The first plant in Denmark is situated at a small lake in Copenhagen that receives water from a brook heavily impacted by CSOs during rain. The plant treats CSO-impacted stream runoff during rain, but switches to treat the lake water (especially for P) during dry weather. The treated water from this lake is the main source of water renewal for the lakes in the centre of Copenhagen. It is expected that treatment of discharges from CSOs and storm sewers using this type of treatment will become popular in the future because the plant is very compact, can start up quickly and has high removal efficiency compared with most other techniques for treatment of transient discharges.

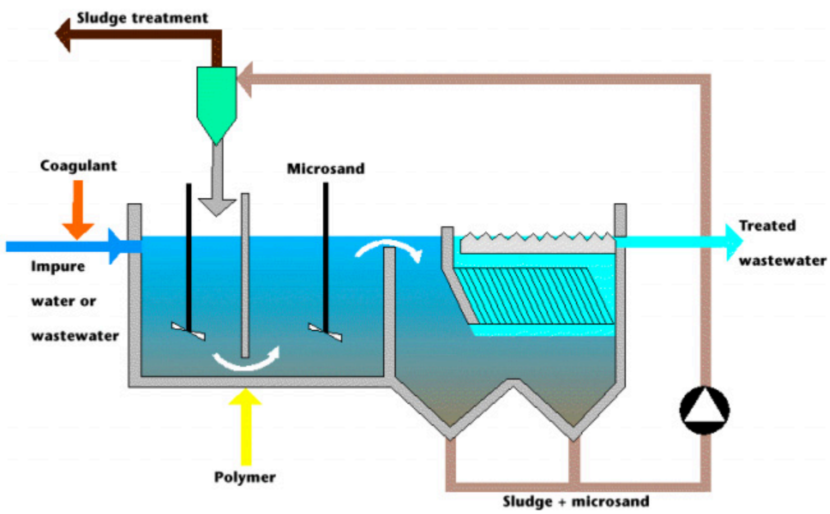

Figure 2. The Actiflo® compact physical-chemical treatment plant.

3.3.3 Combined detention basins and wetland systems

Detention basins at CSOs have generally been constructed so that, once full, overflow is discharged directly into the receiving water. The basin volumes depend on the type and size of the receiving water but the basins are often fairly large and costly. In Herning, Denmark, constructed wetlands are used in combination with conventional detention basins to reduce costs. Basins have been built with storage equivalent to a depth of 2-4 mm of rain. When a basin is full the overflow goes to a more natural, large, wet pond, 25 mm or more, that also receives discharges from separate stormwater systems. From here excess water is then discharged slowly and with delay before entering the stream. In this way the stream is protected against erosion caused by peak flows. The overflow is less polluted because of sedimentation and degradation in the pond, the treatment plant is protected against the large water volumes from emptying basins and the system is much cheaper than building normal concrete basins with the same volume. A monitoring system (oxygen and flow) in the river has shown that the receiving water in the stream can handle the limited load from this system. There are only very few measurements at the inlets and outlets of the ponds, but the available data show a 40-70% reduction of concentration of COD and nutrients, and the concentrations in pond outflow are often lower than the concentrations in treated water from wastewater treatment plants.

3.4 BMPS FOR SOURCE CONTROL

3.4.1 Stormwater harvesting and infiltration

Stormwater infiltration is common practice in residential areas in both Denmark and Sweden using e.g., permeable surfaces, swales and infiltration trenches. Recent findings in Denmark show that infiltration in existing urban areas with combined sewer systems may lead to 30% increased evaporation, 25% increased groundwater recharge and thus 40% reduced runoff via the sewers [21] and that infiltration is much more efficient for combined sewer overflow abatement than conventional detention ponds [22]. Furthermore, although it is difficult to implement on a large scale, stormwater infiltration has now been proven a realistic option even in central urban areas [23]. Difficulties, however, still remain when the soil permeability is too low and when there is risk of contaminating sensitive groundwater reservoirs.

Harvesting of stormwater and use for toilet flushing has received a lot of attention during the past years in Denmark. It appears to be expensive compared with normal water supply, the resource savings are only marginal [24], and rainwater tanks are clearly less efficient than detention basins for CSO abatement [25]. In spite of these facts, however, the public increasingly sees stormwater harvesting for reuse as a secondary water source in households and industry as a 'sustainable' solution.

3.4.2 Snow management

In cold regions, great demands are made on municipalities and their engineers regarding snow handling. Snow handling, such as ploughing and removal, is necessary in order to achieve safe road conditions, but is expensive and has a detrimental effect on the environment. Snow management is becoming an issue even in Danish municipalities due to the increased local focus in some areas on environmental protection.

Optimal snow handling takes the whole process into account, allowing the best practice possible with the lowest environmental impact at the most reasonable cost. A Swedish snow handling strategy [26, 27] requires that cities and towns should be divided into different areas on the basis of the quality of the snow in each area. The snow that is more polluted should then be transported to a snow deposit appropriately located, designed and operated to minimize the negative environmental effects. In contrast, the clean snow can be dumped where stormwater discharges are allowed. If the snow handling strategy is followed, the snow deposits where the dirty snow is dumped will have a significant reduction in the volume of snow, but also a higher content of pollutants, compared to the deposits today.

3.4.3 Street sweeping

In order to evaluate street sweeping as a pollution control measure, street sweeping experiments were performed in Jönköping, Sweden [10]. The test period was in total six weeks; during the first three weeks the street was swept once a week and the following three weeks every workday. Generally, an increasing frequency of sweeping increased the amount of removed sediment. The increased amount of sediment was, however, not proportional to the increase in sweeping effort. It was concluded that street sweeping with modern sweeping equipment can be an effective pollutant control measure.

4. Discussion

4.1 STORMWATER BMP DESIGN AND PERFORMANCE

The examples above illustrate that generally, there is no good overview of what can be obtained using various (structural) BMPs, and that non-structural BMPs are still not widely applied in practice. Treatment efficiencies for structural BMPs are only known for few pollutants and are questionable in some cases. Part of the reason for this is that structural BMP design often neglects the inherent transient nature of extreme stormwater flows; they are almost impossible to treat and it may thus be necessary to by-pass the treatment system for such flows. Bad examples where this has not been realised are for example oil separators and ponds where the retained matter is flushed out during heavy rainstorms. There are a few examples where structural BMPs have been built in a modular way, allowing different treatment steps to operate in sequence (e.g. post-treatment of pond effluent). There are also examples of integrated systems combining e.g., storage and settling with filtration and plant-uptake in vegetated systems. However, authorities seem to lose interest in monitoring programs after investments have been made, thus terminating the measurements before the results of scientific value have been obtained. This makes the results difficult to generalise, and cause-effect relationships that allow BMPs to be designed based on a desired pollution removal do not exist. The only cases where high removal efficiencies have been obtained are for combined storage-treatment systems where compact physical-chemical treatment plants and sorption/filtration systems show promising results. Most practical projects in Denmark aim at reducing the emission of ‘traditional’ pollutants such as N, P and COD, but in Sweden some attention has been given to heavy metals and polyaromatic hydrocarbons. There is some evidence that pollutant concentrations can build up in pond and wetland sediments and biota to critically high concentration levels, and this actually creates a dilemma. On one hand, structural BMPs such as ponds and wetlands are considered public amenities that are integrated actively into modern town planning, but on the other, this may lead to contaminated sites developing in and around urban settlements and to concentrations of hazardous compounds increasing in both flora and fauna. The same phenomenon occurs for stormwater infiltration; pollutants can build up to critically high concentration levels within a few decades, leaving soils unsuitable for vegetable gardening [22]. So far, health hazard due to presence of pathogenic bacteria, viruses and parasites in stormwater runoff has not received much attention, but both theoretical and experimental investigations are under way [1].

4.2 BMP DESIGN FOR IMPACT MITIGATION?

Although some theoretical assessments have been made, there is so far no experimental evidence indicating changes of receiving water quality caused solely by stormwater BMPs. The main problem is that receiving water quality is affected by many other impacts than 'just' stormwater discharges. For example, recent knowledge stresses that the primary factor determining the ecological quality of urban streams is the physical structure of the streams themselves. Thus, impact mitigation needs to be addressed differently by following a stepwise, adaptive, approach [28, 29] where small steps are taken, sometimes employing rather unproven measures, and followed up by adequate monitoring programmes before further action is taken.

4.3 INTEGRATED STORMWATER POLLUTION MANAGEMENT

Integrated management of stormwater and the contained pollutants has so far only been addressed academically in Denmark and Sweden. Based on principles known from environmental chemistry and eco-toxicology for risk assessment of chemicals it has been emphasised that there are basically only three options for action [30]:

- Contaminants can be controlled at the source, e.g., by controlling the use of building materials contributing to erosion and dissolution of contaminants, or by cleaning urban surfaces (non-structural BMPs).
- Contaminants can be immobilised or degraded in structural stormwater BMPs designed for the specific purpose of pollution reduction.
- Contaminants can be discharged in the least harmful manner.

In principle, stormwater and the associated pollution can be discharged to surface waters and sediments or to urban soils and groundwater, depending on how various stormwater BMPs are used. Pollutant mass flows can be quantified using simulation models [31], but the basic question 'where to place the pollution' has still not been answered, and management of hazardous compounds is still not properly integrated into stormwater management.

5. Conclusions

Many structural and non-structural BMPs have been tested in Denmark and Sweden during the last decade; some of them on a large scale and some on an experimental scale. Experiences gained are not consistent, and the actual effects of BMPs on receiving water quality are not known or not understood. From a rational viewpoint, no further large-scale facilities should be constructed until such effects can be determined. However, there are several other driving forces that determine the development. One such a factor is the development of legal instruments and directives, both at a national level and the European Union level. Another important factor is the city planning in many cities, which currently often include "blue-green" areas with visible stormwater. A third driving force is the aim for sustainable solutions within the water and wastewater sector, which also puts demands on the management of stormwater.

6. Acknowledgement

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REDUCTION OF RUNOFF BY MODIFIED TROUGH – INFILTRATION TRENCH SYSTEM – A CASE STUDY OF LESZNO TOWN

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1. Introduction

Stormwater management based on drainage technology using trough-infiltration-trench system is an alternative to the disposal of rainwater by traditional storm sewer systems. Troughs can be used for short-term storage of rainwater, which infiltrates through their bottom to the ground. Unfortunately in practice low soil permeability very often does not allow for sufficient intensity of infiltration. This situation can be solved by coupling the trough with a trench. Stored water is buffered by permeable trench material and released slowly by percolation to recharge groundwater and discharge, under control to the receiving waters. In spite of many advantages over the traditional stormwater systems, a traditional trough-infiltration–trench system cannot be applied in all topographic and construction conditions, which can form a barrier to implementation of the system. The purpose of the paper is to present a retrofit of a traditional trough-infiltration–trench system, which can be applied also in the conditions deemed earlier unacceptable.

2. Basic concept and structure of the system

A traditional trough-infiltration-trench system consists of two basic elements: a trough and infiltration trench (retention layer section) separated by an intermediate layer [1,2]. After the rainfall event, the surface runoff is transported to a grassed trough by gutters and temporarily stored in it. During infiltration through the active soil (humus) layer laid directly under the bottom of the trough, percolating water is treated as a result of partial removal of solids and dissolved chemicals. Treated water infiltrates downstream through the intermediate layer to the trench filled with porous materials (gravel) and wrapped in geotextile, from which it can infiltrate into the ground surrounding the trench, or be released into the storm sewer system. The outflow is controlled by a valve and is extended in time. This system has proved to be one of the more effective best management practices (BMPs) and attempts have been made to apply it for reducing not only the surface runoff but also the runoff from building roofs. However there are some limitations in application of this

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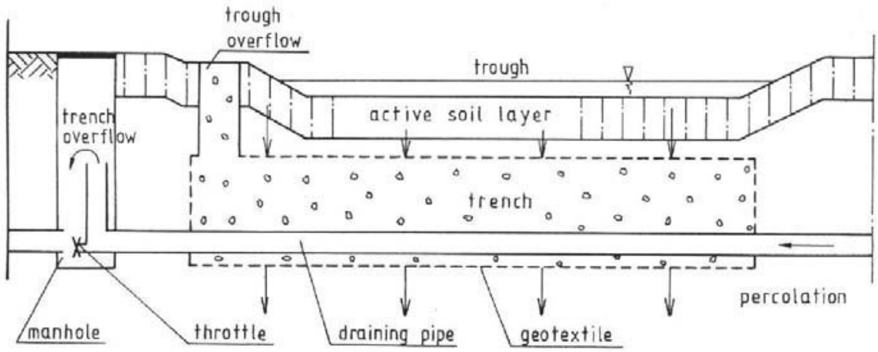
system, which can be difficult to overcome. First of all it is the lack of space in the developed areas where troughs could be located.



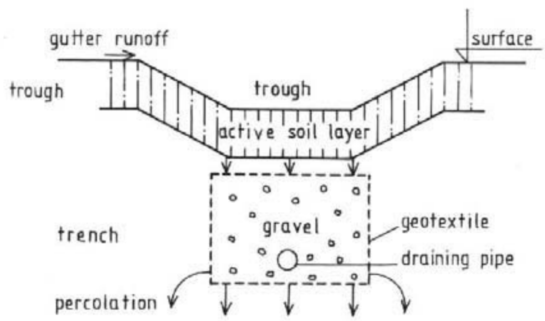


Figure 1 . Scheme of the traditional trough-infiltration-trench system

They should be located in sufficiently large green areas in the neighbourhood of the buildings under consideration. In the case of draining roofs by inner rainwater downspouts, there is another serious argument against application of the traditional trough–infiltration-trench system. These pipes are connected at the downstream ends to horizontal collectors, which are laid in cellars of buildings and below the ground level of the surrounding buildings. The depth of collectors is limited by the depth of ground frost, i.e., h= 0.80 – 1.20 m in Poland, which is much larger than the depth of trough (assumed as 0.30 m). It means that water cannot be delivered to troughs by gravity (except for significant differences in the elevation of ground surface in the vicinity of buildings).

The aim of the paper is to present an application of a modified trough–infiltration- trench system Innodrain, which was developed by F. and H. Sieker [2] and used [3] to reduce runoff from the roofs of a new school in the Town of Leszno, Poland.

3. Background of system modification

The basic assumptions made in modification of a traditional trough-infiltration-trench system can be summarised as follows; two factors, inner rainwater downpipes situated inside buildings and the lack of green areas in the building vicinity necessitated modifying the traditional system. It was replaced by an Innodrain scheme, which is characterised by the following features: troughs have a form of boxes with concrete vertical walls and permeable bottoms elevated about 30 cm above the ground surface and they are arranged in a series (Fig. 2).

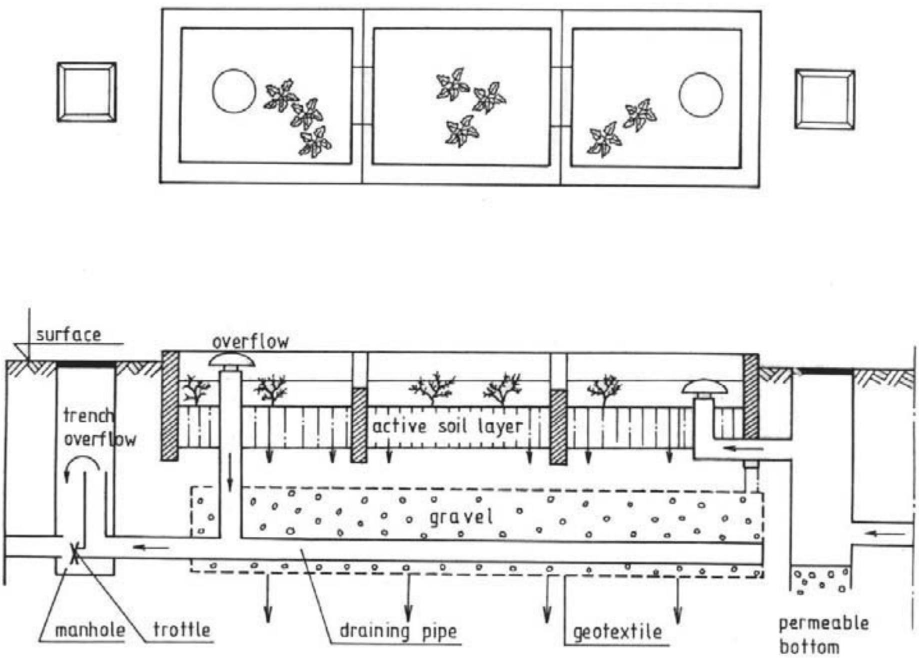


Figure 2. A retrofit of the trough-infiltration-trench system

Special vegetation is planted inside the boxes in order to intensify treatment of rainwater during infiltration through the active upper soil layer and increase aesthetic amenities of the project. Hence the name for these modified troughs was proposed in the paper as *flower boxes*.

Gutters in a traditional system transporting stormwater runoff are replaced by underground collectors (pipes) through which water from the roofs is delivered to the manhole preceding the first flower boxes. If the box is full, the filling of the next box starts via a weir overflow and this procedure is continued for the remaining boxes. In the last box an overflow is situated through which an excess volume of water is spilled from this box directly to a drain in the trench. The main volume of water infiltrates, however, through the active soil and intermediate layers to the trench occupying the lowest position in this sequence. From here, a basic outflow determined by ecological requirements and controlled by a throttle is delivered to the storm sewer network and finally to the receiving waters. An essential volume of water stored in the trench, in the case of favourable conditions (low level of groundwater, high permeability of ground), percolates into the ground. If trench capacity would be exceeded, an overflow is used to spill the excess volume into a storm sewer.

In spite of many advantages listed above, there is a certain problem concerning the implementation of the Innodrain system - a volume of water stored in the upper section of the system consisting of collectors and manholes at the entrance to flower boxes (Fig. 3) can not be removed from the system by any of the above specified ways.

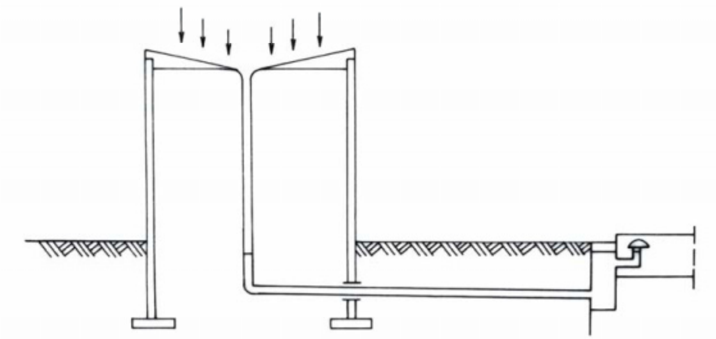


Figure 3. A longitudinal cross-section of rainwater downpipes and their connection to flower box through a collector and manhole

Keeping standing water in flower boxes in dry weather is counterproductive, because it forms an environment for growth of aquatic microorganisms. Furthermore, a piezometric pressure in collectors transporting rainwater from downspouts to flower boxes is determined by the water level in flower boxes and reaches almost the ground level. In the case of leakage, water from this section of the system can inundate the surrounding area. To avoid this drawback, a permeable bottom in the manhole at the entrance to flower boxes (through which water percolates and is removed from collectors) was proposed by the authors of the Innodrain system. The stored water can infiltrate this way to the ground after the end of runoff. It should be noted that the process of infiltration through the bottom of manhole continues during the whole period of runoff, but this volume is not significant in comparison to the volume of total outflow from flower boxes and can be neglected in the design of the system.

4. A case study

4.1. A STUDY OBJECT

As a study object a new school in the Town of Leszno (70 km south from Poznan, Poland) was chosen. It consists of several connected buildings [4] forming a layout similar to the letter U (Fig. 4). Runoff from roofs is directed to 30 inlets connected to inner rainwater downpipes (diameters 160 and 200 mm), which drain into a network of collectors in the cellars of the buildings. They are connected to main collectors passing by the walls of buildings in 5 points at the depth ranging from 1.00 to 1.80 m and delivering water to the manholes preceding the Innodrain structures.

4.2. HYDRO-GEOLOGICAL CONDITIONS

Hydro-geological conditions under the school studied are favourable for any type of the trough–infiltration-trench system. Based on samples taken from 8 geological drill holes 6 m deep it was found that below a 0.5 m thick surface layer of soil, up to 6.0 m of fine sand is deposited with intrusions of clay sand and medium sand. The groundwater table was observed at the level of 4.8 - 5.0 m below the ground surface and it is expected that seasonal fluctuations should not exceed 1.0 m. The average value of the coefficient of permeability was determined to be within the range from 0.01 to 0.001 m/s.

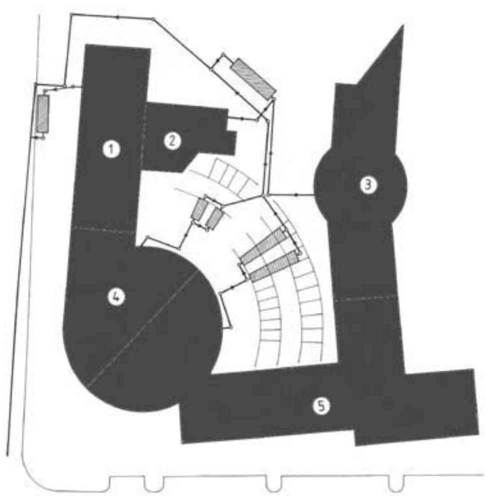


Figure 4. A plan view of the case study area – the school in Leszno with Innodrain structures

4.3. RAINFALL ANALYSIS

The rainfall analysis was used to derive a formula allowing computing the maximum rainfall depth in time T with a given probability of occurrence p (or frequency n). It was assumed [3] that the annual maximum rainfall depths may be described by the Weibull 3-parameter distribution. Hence a quintile of maximum rainfall depth in time T with the probability of occurrence p, can be computed [5] from a formula

$$P_{max}(T,p) = \varepsilon(T) + \alpha(R,T)\cdot(-\ln p)^{1/\lambda} \tag{1}$$

where: $\varepsilon(T)$ – a lower band parameter, which depends on rainfall duration time and can be computed from equation

$$\varepsilon(T) = 1.42\cdot T^{0.33} \tag{2}$$

$\alpha(R,T)$ – a scale parameter, depends on the region of Poland R and rainfall duration T; for a central region in which the Town of Leszno is situated it can be computed, depending on time T, from one of two equations
for 5 min ≤ T ≤ 2 hours

$$\alpha(R,T) = 4.693\ \ln(T+1) - 1.24 \tag{3}$$

for 2 h ≤ T ≤ 18 hours

$$\alpha(R,T) = 2.223\ \ln(T+1) + 10.639 \tag{4}$$

where λ is a shape parameter.

The ratio of rainfall depth and duration is defined as the rainfall intensity of a given duration and frequency n (a reciprocal of probability of occurrence p), hence

$$v_{T(n)} = \frac{P_{max}(T,p)}{T} \qquad [mm/min] \tag{5}$$

where:

$P_{max}(T,p)$ – quintile of the maximum rainfall depth over time T, with probability of occurrence p [mm] ; T – time of rainfall duration [min].

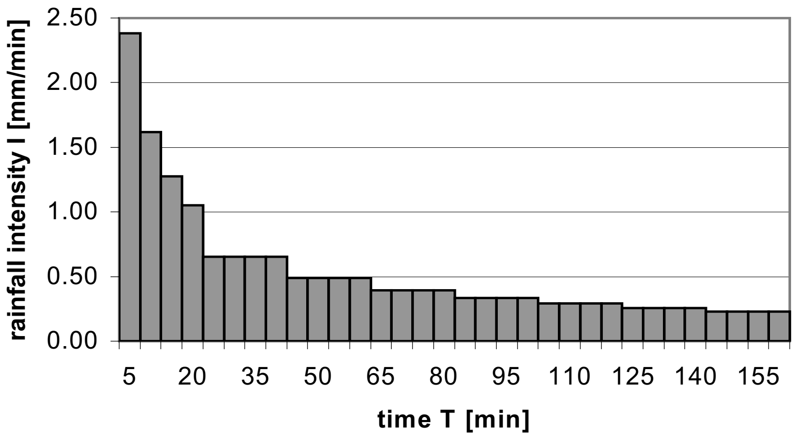


Figure 5. Hyetograph developed for the region of Leszno (exceedance probability p = 20%)

For given time interval T, e.g., T=5 min, a hyetograph representing distribution of rainfall intensity in time can be produced using the above equations (Fig. 5). In the absence of historical rainfalls series, it can be treated as a synthetic hyetograph and used for the design of the trough-infiltration-trench system.

4.4. DESIGN PROCEDURE

In the first stage of system design roofs of school buildings were divided into five partial watershed areas (Fig. 3), from which water should be directed to separate elements of the Innodrain system (including *flower boxes* and *trenches*). The criteria for location of these structures were chosen taking into consideration the following priorities:

- a short length of collectors transporting water from the building to the Innodrain elements,
- easy connection of drains collecting some volume of water from these elements to the storm sewer network and the receiving waters,
- ownership of land where Innodrain structures are expected to be located, and
- compliance with the existing plans land use plans.

In the case of two contributing areas (No. 2 and 5), the outflow can be directed to one Innodrain element and, therefore, the total number of elements was reduced to four. Their location was combined with determination of the surface area of component elements. It is recommended [2] that the area of troughs should be about 5% of the drainage area. These values obtained in the project did not exceed the upper limit (Table 1).

TABLE 1. Surface characteristics of partial drainage areas and troughs

Drainage area number	1	2	3	4	5
Drainage area [m^2]	970	400	1310	1030	2890
Combined drainage area [m^2]		1710			
Trough area [m^2]	30.0	42.0		36.0	72.0
Ratio $\frac{\text{trough area}}{\text{watershed}} \cdot 100$ [%]	3.1	2.4		3.5	2.5

Troughs of the Innodrain system are constructed in the form of concrete boxes (*flower boxes*), hence their width is usually fixed (it was assumed 3.0 m in the case of Leszno) and only their length varies. They can be arranged in a series or in a mixed way (two or more series of boxes combined in parallel) depending on the available space. In the case of the school in Leszno, three Innodrain elements were situated in the courtyard [3] (in order to comply with the existing plan of land use [4], two with flower boxes connected in parallel, and one in series), one in front of the building – on the front lawn (with flower boxes arranged in series). The final step in the design procedure is connected with sizing the trenches. Because their depth is usually assumed (0.5 m in the case of the Leszno school), the width is equal to the width of trough (3.0 m), and the only unknown is its length. It is determined on the basis of continuity equations for the elements of the Innodrain system. Due to a large volume of calculations, a computer package MURESIM developed by Ingenieurgesellschaft Prof. Dr. Sieker [2] is very helpful in executing these computations. It was applied also in this case and used to determine the third parameter for each trench. They are presented in Table 2.

TABLE 2. Surface dimensions of troughs (flower boxes) and trenches

Trough number	1	2 & 3	4	5
Trough area [m^2]	30.0	42.0	2*18=36.0	2*36=72.0
Trough dimensions:				
- width [m]	3.0	3.0	3.0	3.0
- length [m]	10.0	14.0	2*6=12.0	2*12=24.0
Trench length [m]	10.0	15.0	2*6=12.0	2*15=30.0

4.5. ANALYSIS OF RESULTS

Analysis of the data in Tables 1 and 2 allows formulating two essential observations: Trough area is influenced by the surface cover of the drainage area and limitations concerning the location of flower boxes, e.g., drainage areas Nos. 1 and 4 are similar, but their trough surfaces are significantly different (by 20%). There is a possibility to decrease the trough capacity by decreasing its surface area significantly below the recommended 5% of the contributing drainage area, but at the cost of increasing the volume of the trench underneath by extending its area (length), e.g., as was the case of the drainage area No 5.

5. Concluding Remarks

(a) Modification of the classical trough-infiltration-trench system, applied in conjunction with the Innodrain system, allows reducing runoff from roofs of buildings in densely populated downtown areas by incorporating small green areas equipped with a system of inner rainwater downspouts.

(b) The main modifications of the classical trough-infiltration-trench system consist in:

- gutters transporting runoff from rainwater downpipes to the trough are replaced by underground collectors,
- troughs, which in the traditional solution are excavated in the ground are replaced by a series of concrete boxes (*flower boxes*) with permeable bottom.

(c) If the historical rainfall data are lacking, a synthetic hyetograph representing precipitation for the studied region was proposed as an alternative input to the model.

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THE KINGSTON POND: A CASE STUDY OF STORMWATER POND UPGRADING

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1. Introduction

The use of stormwater ponds in urban drainage systems in Canada and the United States dates back to the 1960s. In Ontario, the first ponds were built in the metropolitan Toronto and Ottawa areas. Flood reduction and drainage cost reduction were identified as the two most important objectives; control of stormwater pollution was ranked fairly low [1]. By 1981, there were almost 13,000 drainage storage facilities in Canada and the US [2].

Although the first publications on the quality of urban runoff and its impact on receiving waters appeared in the 1960s (e.g., Weibel [3]), many more were published during the following two decades and this period of research culminated with the 1983 publication by the US EPA of the "Results of the Nationwide Urban Runoff Program" [4]. At the same time, beginning in late 1970s and into the 1980s, the use of stormwater ponds for water quality enhancement was investigated and pond processes contributing to such an enhancement were studied. For example, Whipple [5] proposed a dual-purpose detention basin, Whipple and Hunter [6] studied the settleability of urban runoff pollution in ponds, and Hvitved-Jacobsen et al. [7] investigated the fate of phosphorus and nitrogen in ponds.

A generally advanced understanding of urban stormwater pollution in the mid to late 1980s and early 1990s led to the promulgation of government policies aimed at the control of stormwater pollution and the development of a wide range of "best management practices", which included extended detention basins. Schueler [8] and Ontario MOEE [9] contain design guidelines for such basins.

Retrofitting for water quality improvement was first advocated in the early 1990s [10]. Marsalek et al. [11] were among the first to point out that "older ponds needed to be examined in the light of new environmental knowledge and objectives". They also noted that some of the earlier ponds, developed for flood control only and sometimes poorly maintained, may adversely impact on the environment and represent potential liabilities, a warning also made earlier by Jones and Jones [12].

The Kingston Pond, constructed in 1982 to reduce post-development peaks to pre-development levels, is typical of thousands of ponds built in southern Canada and the United States during the pre-stormwater quality management era. As such, it

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is an excellent candidate for retrofitting. To determine appropriate retrofit designs for water quality improvement and to provide guidance for the design of extended detention ponds, science-based laboratory and field studies of the Kingston Pond performance, stormwater pond processes, and effluent polishing were conducted.

The objectives of this paper are to provide an overview of the Kingston Pond retrofit, to summarise the studies of the Kingston Pond performance and processes, and to describe the Kingston Pond retrofits addressing the identified inadequacies.

2. Kingston Pond: History and Research Program Summary

The Kingston Stormwater Pond, an on-line stormwater management pond on the west branch of the Little Cataraqui Creek in Kingston, Ontario, Canada, was built in 1982 to reduce peak flows from the parking lot of a shopping mall. The two-stage pond consists of a permanent wet pond (area 5,200 m^2 and 1.2 m average depth) and a dry pond (area 5,000 m^2) that floods during larger storm events. Figure 1 shows the pond layout and the location of instrumentation and weirs at the inlets and outlet. The pond currently receives runoff from the upstream suburban/rural area and from the shopping mall parking lot. The upstream catchment, with an area of 4.54 km^2, has developed over time so that a significantly larger area is now paved, and directly or indirectly connected to the drainage system and pond.

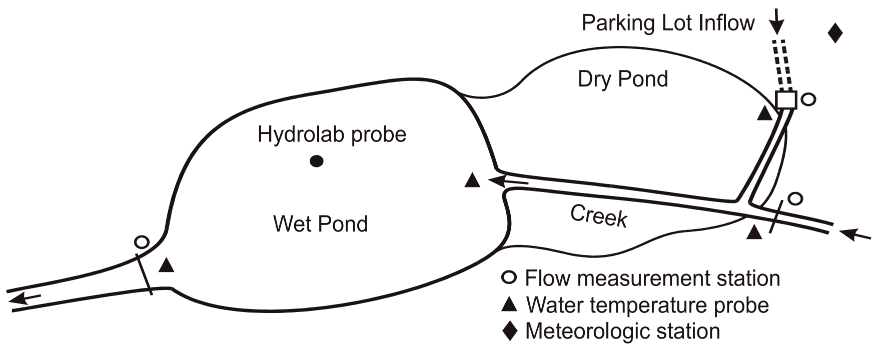

Fig. 1. Kingston Stormwater Management Pond

Long-term interdisciplinary research was conducted at this full-scale stormwater treatment facility for a decade with the objective of understanding how this particular facility functions in terms of quality and quantity control, and with the goal of proposing cost-effective solutions to improve these functions in under-designed and overloaded systems. Research findings on the Kingston Pond performance and processes are summarised in the following paragraphs; detailed results are given in the publications cited.

2.1 PERFORMANCE - FIELD SEASON

Van Buren et al. [13,14] presented a methodology for assessing the pollution control performance of an on-stream pond and applied the methodology to the Kingston

Pond. The assessment is based on constituent mass balances for both baseflow and event conditions. Results on pollutant removal rates, which were based on data collected over two field seasons, were provided for selected dissolved constituents, nutrients, suspended solids, metals and organic contaminants. In summary, dissolved constituents exhibit zero removal during baseflow periods and positive removal for events; nutrients and suspended solids exhibit negative removal for baseflow periods and positive removal for events; and metals and organics exhibit positive removal for both baseflow periods and events. In terms of field season performance, contaminant removals during storm events were fair to good, but the pond is now undersized because of significant upstream development after 1982. Moreover, during baseflow periods, the pond exports contaminants.

2.2 PERFORMANCE – WINTER

Marsalek et al. [15,16] characterised the winter operation of the pond based on field studies over two winters. The pond froze over in late November and ice thickness varied from 0.2 - 0.5 m. The measured and modelled velocity field indicated a fast flow region, a small dead zone and a large recirculation zone. During a snowmelt event, near-bottom velocities reached 0.05 m/s, but were not sufficient to scour the fine bottom sediment. Pond water temperature increased with depth from 0.5 °C to 3.5 °C. High dissolved oxygen (DO) levels (6-12 mg/l), which were generally observed throughout the pond, indicated stable aerobic conditions at the sediment-water interface. In one brief episode, DO fell to zero after a long cold spell. Reduction in DO readings from inlet to outlet indicated an oxygen consumption of about 1.7 kg/day. pH ranged from 7.1 to 8.9. Conductivity readings indicated large amounts of total dissolved solids (TDS), representing mostly chlorides from de-icing agents. During baseflow, conductivity increased with depth (TDS concentrations up to 1200 mg/l near the bottom). Average trace metal concentrations were mostly below detection limits. The study of winter performance and processes showed a cycle of chloride accumulation during winter and export during spring and summer.

2.3 SEDIMENT ACCUMULATION

Marsalek et al. [17] reported that pond bottom sediments had accumulated at an average rate of 0.02 m/year and comprised gravel, sand, silt and clay; the gravel and sand accumulated only by the inlet whereas the silt and clay were spread throughout the pond and represented up to 45 % and 54 % of the total sediment respectively. Marsalek [18] estimated the volume of the inlet sand spit at 150 m^3, and the corresponding sediment mass 160 t, accumulated over 15 years. The water content of the sediment (by volume) ranged from 48 % by the inlet to 75 % at the outlet. The assessment of total metals in the sediment against the Ontario Ministry of the Environment (MOEE) sediment quality guidelines indicated a gross pollution by Cr and marginal to significant pollution by Cd, Cu, Fe, Pb, Mn, Ni, and Zn. Sequential analysis of the samples indicated that 40-90% of the retained metals were in potentially mobile forms. The chemistry of the suspended particulate, representing silt and clay, was similar to that of the bottom sediment, except in the case of Cr. They noted, “the results indicate a significant disadvantage of on-stream stormwater ponds built on urbanizing catchments – such ponds tend to accumulate sediment at relatively high rates and will require more frequent sediment removal than off-line facilities".

Krishnappan et al. [19] conducted three surveys of suspended solids at 17 points in the pond. Observed suspended solids were composed mainly of flocs, with maximum sizes ranging from 30 to 212 μm for winter and summer surveys respectively. Using an empirical relation developed for floc fall velocity, they determined that the highest floc settling velocities were for flocs of size 5 to 15 μm; larger flocs would settle more quickly, but may be broken up by flow turbulence into smaller fragments.

2.4 VELOCITY FIELD

Shaw et al. [20] determined from field measurements and computer simulations that the flow pattern in the pond is very dynamic and complex – the complexity resulting from wind stress on the pond surface. A combination of high wind and low inflow generates a circulation pattern that is pronounced in the vertical, whereas with relatively low wind stress, the velocity field is determined by inflow momentum and pond geometry and is characterised by circulation in the horizontal plane. In general, the velocity field measurements displayed a jet-type flow through the pond with associated dead zones and recirculation zones. Regardless of the magnitude of the inflow, the length to width ratio of the pond (1.5:1) and the inflow momentum promote short-circuiting of the flow and limit the efficiency of the settling.

2.5 INTERNAL BAFFLES

Matthews et al. [21] reported on an interim retrofitting measure whereby installation of strategically placed baffles in the pond increased the length-to-width ratio of the flow path from 1.5:1 to 4.5:1. Results of dye-tracing studies performed after the retrofit demonstrated an increase in retention times with a reduction in the speed and volume of short-circuited flow and a decrease in wind-generated flow patterns due to the baffles. The hydraulic efficiency of the pond (defined as the ratio of measured to volumetric retention times) increased from 0.65 to 0.85. They inferred an increase in pollutant removal through sedimentation processes from a comparison of retention time distributions before and after baffle installation.

2.6 RESUSPENSION AND SEDIMENT EXPORT

Watt [22] investigated the effects of the operation of a decorative fountain on the total suspended sediment (TSS) concentrations in the pond and in the pond outflow. Analysis of continuous measurements of TSS over periods when the fountain pump was on and off showed that outflow concentrations "jumped" by about 100 mg/l when the pump was turned on during baseflow conditions. Currents generated by the operation of the pump were resuspending pond bottom sediment and thereby contributing to sediment export during baseflow.

2.7 EFFLUENT POLISHING BY CONSTRUCTED WETLANDS

Rochfort et al. [23] described the performance of field-scale subsurface flow constructed wetlands that received a portion of the pond effluent. It was generally found that acceptable removal of suspended solids, soluble metals and phosphorus occurred, while organic carbon was not removed effectively (possibly due to low loadings during the test period). The main removal mechanisms appeared to be biological assimilation and, to an unknown extent, physical adsorption within the limestone medium. They concluded that the subsurface flow wetland system could be used in conjunction with extended ponds and surface flow wetlands in a multiple pond design, or by itself provided that adequate treatment of solids occurs.

2.8 EFFLUENT POLISHING BY BIOFILTER

Anderson et al. [24] describe the laboratory and initial field studies associated with the testing of a field-scale submerged aerobic biological filter (SABF) for polishing effluent from the pond. The SABF unit demonstrated the ability to remove organic carbon (10-20%), suspended solids (90%), and ammonium nitrogen (60-95%) depending on influent loading and hydraulic residence time (HRT). Phosphorus was removed in the lab experiments, but field results were inconclusive, due mostly to the low loadings. Copper and zinc were removed in both lab and field filters (27-66%).

Mothersill et al. [25] describe tests conducted on the SABF after one and three years of operation to evaluate the impacts of accumulated sediment on its performance and treatment efficiency. They found that the high accumulation of sediment, predominately in the upper 200 mm of the filter, changed the hydraulic properties of the biofilter and decreased the effectiveness of aqueous carbon and phosphorus removal. The backwashing system was not effective in sediment removal, in part because of limited head and also because of higher than usual sediment loadings during the test period because of construction activities in the upstream catchment and the operation of a decorative fountain in the pond.

2.9 THERMAL BALANCE

Van Buren et al. [26] describe the development and assessment of the thermal energy balance for the Kingston Pond. The energy balance method was used successfully to predict average pond temperatures. During dry-weather periods, the pond temperature increased as a result of solar heating, and thermal energy stored in the pond accounted for about 2% of the total thermal energy. In contrast, during wet weather periods, pond temperatures decreased as a result of limited solar radiation and replacement of warmer pond water by cooler inflow and thermal energy supplied from the pond accounted for almost 3% of total thermal energy. Lack of tree canopy surrounding the pond and the inlet channel provides little shading and increased opportunity for solar heating. The on-stream nature of the pond promotes increased temperatures of receiving waters during dry-weather periods because the cooler baseflow is continually heated during its residence time in the pond.

3. Kingston Pond: Operation and Maintenance Concerns

During the period of research studies summarised above, it became apparent that operation and maintenance of stormwater management facilities and the perceptions of multiple stakeholders about what these systems are intended to accomplish will play integral roles in the success or failure of stormwater management facilities in fulfilling their primary function of ecosystem protection. These factors are also significant in the development and implementation of retrofit strategies. Watt et al. [27, 28] present operation and maintenance concerns for each of three stages: initial design, regular operation and maintenance, and retrofitting with examples drawn from experience at the Kingston Pond. They link these examples to conflicting expectations of key stakeholders. Anderson et al. [29] review both the research results and the overall experience at the Kingston Pond and conclude that there are a number of identifiable factors, termed critical issues that will significantly influence the success, failure and sustainability of stormwater facilities. They group these factors within the categories of initial design, operation and maintenance, performance, and adaptive design, which includes retrofits.

4. Kingston Pond Retrofits to Address Inadequacies

4.1 OVERVIEW

At the beginning of the research program, it was clear that the pond required retrofitting in order to improve water quality control. What was not clear, however, and accordingly required demonstration, were the performance of the existing pond and quantification of problems resulting from its inadequacies. Five principal Kingston Pond inadequacies were identified: (i) it does not have a sediment forebay, (ii) it is on-line, (iii) its small length/width ratio yields HRTs that are too short, (iv) it has a decorative fountain that resuspends sediment, and (v) it is not adequately maintained. In addition, it does not remove dissolved contaminants and cannot remove the fine sediment by sedimentation because of wind-generated currents and associated turbulence. In order to rectify or compensate for these inadequacies, six retrofit strategies are presented: (i) remove the fountain to eliminate resuspension by the pump, (ii) add a sediment forebay to remove the sand particles before they enter the pond, (iii) dredge the bottom sediment to maintain the storage volume, (iv) divert the creek to take the pond primarily offline so that clean creek water does not displace dirty pond water, (v) add internal baffles and modify the outlet to increase HRT, and (vi) add a constructed wetland, or a biofilter, downstream of the pond to remove dissolved pollutants and very fine particulates (Fig. 2).

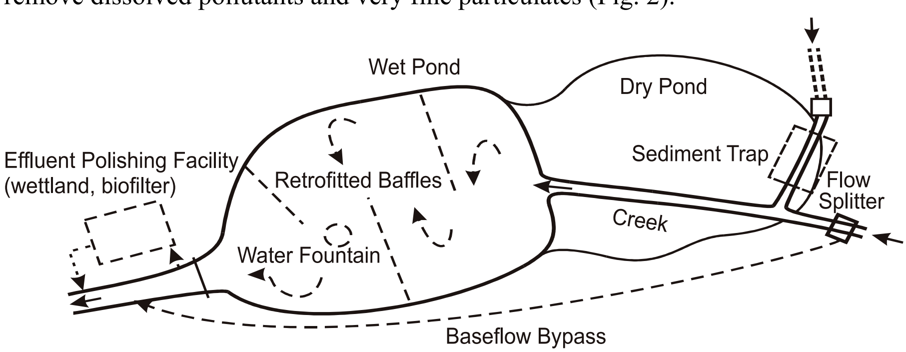

Fig.2. Kingston Stormwater Management Pond: Proposed Retrofit Measures

In each of the following sections, the retrofit is identified by the section heading and a common format is followed: (i) reason for the inadequacy, (ii) problems caused by inadequacies, (iii) description and status of the retrofit, and (iv) predicted improvements, primarily in water quality, but also flood protection and erosion reduction.

4.2 REMOVAL OF DECORATIVE FOUNTAIN

The decorative fountain was placed in the pond because of a lack of communication between the pond designers and the pond owners who placed responsibility for the pond in the parks and recreation department. As noted above, operation of the fountain resulted in a significant export of sediment from the pond bottom to downstream receiving waters. The retrofit has been accomplished in that the fountain has been removed, partly because of environmental concerns and partly because of financial considerations. The primary benefit of the retrofit is a reduction in total suspended sediment load to downstream receiving waters.

4.3 ADDITION OF SEDIMENT FOREBAY

As indicated in section 2 above, the pond was constructed as a peak-shaving pond and, as was the case with almost all flood-control ponds of this era, a sediment forebay was not provided. As a result, sand accumulated in a spit at the entrance to the wet pond. As indicated in section 2.3, the volume of this sand was measured as 150 m^3 and the accumulation rate was estimated as 10 m^3/year [19]. There are two problems with this situation. First, the sand is reducing the storage volume of the pond and hence reducing its peak-shaving capability. Second, dredging the sand from this location is much more difficult and expensive than for a sediment forebay.

A sediment forebay, 18 m wide and 30 m long, was added to the pond system in 1998 (see Fig. 2) as the second element of a series of retrofits. The forebay is upstream of the confluence of the outflow channel from the shopping mall and the creek and hence traps sediment originating in the shopping mall primarily during winter sanding operations. The expected clean-out frequency is 7-9 years. The primary benefits of the forebay retrofit are maintenance of the flood control ability of the pond and a lower cost of sediment removal.

4.4 DREDGING OF BOTTOM SEDIMENT TO RESTORE STORAGE

As indicated in section 2.3, sediment is accumulating on the bottom at a rate of 0.02 m/year and the pond has never been dredged. As a result, pond storage for flood control has been reduced. The municipality has now recognised the problem and sediment removal is scheduled for November 2003. The primary benefit will be flood control. However, as long as the pond remains an on-stream pond, there will be an additional benefit in that there will be less bottom sediment to resuspend by wind-generated currents and hence less sediment exported during baseflow periods. Finally, in view of potential mobility of metals adsorbed to bottom sediments, the risk of metal release is reduced by sediment removal.

4.5 CREEK DIVERSION TO TAKE POND OFF-LINE

As indicated in section 2, the pond was constructed in 1982 as a peak-shaving pond to reduce post-development peak flows from a shopping mall parking lot to pre-development levels. It was designed as an on-line pond. Because the upper catchment was largely undeveloped and its future development was not an issue in the design, whether the pond was on-line or off-line was of no significance in the sizing for peak shaving capability. The decision to go on-line was governed by cost considerations, that is, the proximity of the shopping mall outflow conduit to the creek.

The on-line nature results in four problems. First, the upstream catchment has been allowed to develop with only limited stormwater management. As a result, post-development flows from this area are well in excess of pre-development flows. The pond is too small to control the flows from both the shopping mall parking lot and the upstream catchment. Second, sediments originating in the upstream catchment are reducing the storage capacity of the pond faster than would be the case if only sediments from the mall parking lot were depositing. Third, the pond is too small to be an effective, extended detention pond for both the mall and the upstream catchment. Finally, the relatively clean baseflow from the upstream catchment compromises the pond performance by displacing "dirtier" pond water after storms.

The proposed retrofit involves a diversion whereby creek flows would be diverted around the pond by way of a constructed, mobile bed, open channel. Two alternatives are considered, depending on the status of upstream stormwater management facilities. If upstream flows will be completely controlled by an extended detention pond, then the diversion would be sized to convey a baseflow of about 0.05 m^3/s. In the event that there is no extended detention pond built upstream, then the diversion would be sized to convey peak flows in the range 5 to 10 m^3/s. The first alternative is preferable from an aesthetic viewpoint in that the low flow channel, which would convey a relatively constant flow, would be more attractive than a large channel, empty most of the time, especially in the park setting of this pond. Accordingly, only the low flow channel alternative will be considered.

The primary benefit of the diversion will be a reduction in TSS exported from the pond and, accordingly, an improvement in receiving water quality. A second important benefit would be a reduction in sediment accumulation in the pond by as much as 25-50 % and hence a corresponding reduction in the annual maintenance cost. A third benefit would be a reduction in downstream flooding due primarily to the provision of upstream control.

The municipality has given no indication that it would consider such a retrofit.

4.6 BAFFLE INSTALLATION TO INCREASE LENGTH/WIDTH RATIO

As noted above, the length-to-width ratio of the existing pond (1.5) does not meet guidelines and results in short-circuiting and dead zones. The likely reason for this inadequacy is that the pond was built in 1982 and the designer had limited knowledge of optimum pond dimensions at this stage of stormwater pond development. We recommend installation of baffles similar to those described in section 2.5 whether or not the pond is taken off-line. The primary benefit would be increased pollutant removal by sedimentation, but another benefit would be a reduction in wind-driven currents and associated sediment resuspension and export.

The municipality has given no indication that it would consider such a retrofit.

4.7 OUTLET MODIFICATION TO INCREASE HYDRAULIC RETENTION TIME

As noted above, even with baffles the HRTs do not meet guidelines and are much too low for effective sediment removal. If the pond is taken off-line, it should be reconfigured as an extended detention pond. Accordingly, the outlet control should be modified to include a low-level outlet as well as an overflow weir. The primary benefit would be increased pollutant removal by sedimentation.

The municipality has given no indication that it would consider such a retrofit.

4.8 EFFLUENT POLISHING BY CONSTRUCTED WETLAND

As noted above, the pond was designed and built as a single stormwater treatment facility. Addition of a sediment forebay has added another element in the treatment train and will remove coarse sediments originating in the mall parking lot before they enter the pond. Taking the pond off-line would eliminate the input of fine sediments from upstream and conversion to an extended detention pond, and installation of baffles would enhance removal of suspended sediments originating in the mall parking lot. Addition of a constructed wetland as a polishing device would complete the treatment train for this system. The primary benefit would be enhanced removal of fine sediments and removal of soluble metals and phosphorus.

The municipality has given no indication that it would consider such a retrofit.

5. Concluding Remarks

The results of a decade of interdisciplinary research and our relations with stakeholders at the Kingston Pond stormwater management facility provide an excellent background for recommending retrofits that are likely to be effective in removing the priority pollutants of today. However, there are two challenges to implementation. First, the community, as represented by it elected officials and public employees, may not deem stormwater pollution as a priority problem and will not authorize the required funding. Second, care must be taken to make the retrofitted facility as adaptive as possible so that future retrofits are possible if conditions change.

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CONTROL OF URBAN RUNOFF STORMWATER DISCHARGE TO RECEIVING WATERS USING OFF-LINE STORAGE

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1. Introduction

Since the 1970s, many studies of stormwater quality have been carried out all over the world proving that concentrations of pollutants in stormwater runoff can exert negative impacts, both acute and chronic, on receiving waters. During the last thirty years, several programs gathered and analysed a significant quantity of data on the quality of wet weather flow discharges permitting the creation of useful databases. In the United States, runoff quality studies were conducted under the USEPA Nationwide Urban Runoff Program (NURP) [1] between 1978 and 1983, the United States Geological Survey (USGS) activities, and in the monitoring program required under the National Pollutant Discharge Elimination System (NPDES). In Europe, the most comprehensive database is the QASTOR database [2], which includes data collected in France since 1970 in several catchments. In Italy, for the time being, there is not yet a uniform and methodical database, although several experimental catchments, such as Picchianti (Livorno) [3] served by separate sewers, or Cascina Scala (Pavia) [4, 5], Fossolo (Bologna) [6, 7], Parco D'Orleans (Palermo) [8] served by combined sewers, are still being monitored.

It is clear that stormwater runoff quality depends on several factors, such as the climate (therefore studies in many parts of the world are necessary), land use, population density and the type of the sewer system (combined or separate).

Furthermore stormwater quality data collected in the same experimental catchment vary from one storm event to another, because of the characteristics of the event, such as the rain depth, intensity and duration, and the antecedent dry days. For instance, Table 1 presents the main characteristics of eight storm events observed in the Cascina Scala catchment, while Table 2 summarises the water quality for the same events, i.e., event mean concentrations (EMC) and maximum concentrations occurring during the events of total suspended solids (TSS), chemical oxygen demand (COD) and biochemical oxygen demand (BOD_5).

Finally, with reference to a single storm event, the concentrations of pollutants are greatly variable throughout the rainfall runoff. For instance, Table 3 presents for an event the pollutants concentrations of all the collected samples, the event mean concentration (EMC) and the mean concentration of the runoff corresponding to the first 5 mm of meteorological net rainfall (MC 5 mm). The pollutants considered are the following ones: chemical oxygen demand (COD), biochemical oxygen demand

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(BOD_5), total suspended solids (TSS), settleable solids (Set S), total nitrogen (TN), ammonium nitrogen (AN), total phosphorus (P), total lead (Pb) and total zinc (Zn). The comparison between EMC and MC 5 mm shows clearly the "first flush" phenomenon, for which the pollutant mass is more concentrated in the first volume of urban runoff water.

Table 1. Main characteristics of rainfall for observed storm events.

Event No.	Date	rain depth (mm)	rain duration (min)	mean intensity (mm/h)	maximum flow(*) (l/s)	mean flow(*) (l/s)	runoff volume(*) (m^3)
1	23/06/00	16.4	108	9.1	555	111	793
2	08/07/00	7	50	8.4	329.5	64	246
3	10/07/00	11	64	10.3	380	58.5	488
4	13/03/01	3.8	247	0.9	36.9	12	194
5	17/03/01	26.2	478	3.3	284.7	51	1536
6	28/03/01	18.6	443	2.5	266.8	35	1055
7	10/04/01	8.4	110	4.6	161.3	54	479
8	20/04/01	15.8	380	2.5	260.7	35	973

(*) referring to combined flow (mean sanitary flow is 4 l/s).

Table 2. Water quality parameters for observed storm events.

Event No.	Date	dry days	TSS		COD		BOD_5	
			EMC (mg/l)	max. conc. (mg/l)	EMC (mg/l)	max. conc. (mg/l)	EMC (mg/l)	max. conc. (mg/l)
1	23/06/00	10.8	364	890	346	1950	163	765
2	08/07/00	3.3	1430	2080	1120	2101	318	550
3	10/07/00	1.8	242	1000	168	658	54	380
4	13/03/01	0.3	- (*)	840	- (*)	1152	- (*)	600
5	17/03/01	3.9	108	1280	124	1642	64	880
6	28/03/01	11	120	2360	139	2434	103	1780
7	10/04/01	3.3	420	1420	545	2040	244	900
8	20/04/01	0.0	377	1190	435	4526	203	2120

(*) samples temporal distribution does not allow a correct evaluation of EMC values.

Table 3. Water quality parameters for the storm event number 7

Sample No.	COD (mg/l)	BOD_5 (mg/l)	TSS (mg/l)	Set S (ml/l)	TN (mg/l)	AN (mg/l)	P (mg/l)	Pb (mg/l)	Zn (mg/l)
1	1176	540	630	27	62.8	26.2	4.92	0.14	0.65
2	1637	750	950	51	56.4	25	7.29	0.15	0.83
3	2040	900	1200	74	65.8	20.2	7.6	0.19	0.84
4	1392	620	910	74	54.7	19	5.4	0.11	0.74
5	1334	520	850	33	64.1	12.9	4.7	4.16	0.79
6	1800	850	1420	75	62.2	11.6	7.15	0.13	0.91
7	1368	660	1010	53	75.1	10	4.94	0.12	0.62
8	761	360	670	23	63.1	10	3.06	0.086	0.54
9	854	380	720	31	44.5	10.3	3.99	0.089	0.56
10	562	260	420	19	39.1	9.5	2.67	0.063	0.55
11	346	165	210	7.5	38.1	12.3	1.93	0.066	0.69
12	194	90	170	4	31	10.7	1.25	0.048	0.34
13	307	140	270	11	28.4	6	1.62	0.064	0.56
14	274	125	250	9	22.9	9.2	1.14	0.045	0.43
15	149	55	170	5	18.8	8	0.65	0.063	0.33
16	142	65	120	5	11.9	6	0.79	0.032	0.14
EMC	545	244	420	-	33	9	2	0.34	0.48

(mg/l)									
MC 5mm (mg/l)	810	365	598	-	45	10.5	3	0.56	0.61

2. Structural measures for stormwater runoff control

Recognising the negative impact of stormwater discharges on receiving waters, a solution to this problem could be found by implementing structural measures (sewer overflows and storage basins) according to different possible schemes:

- sewer overflows only,
- on-line storage basins,
- sewer overflows coupled with off-line storage basins.

Figure 1 presents, for both combined and separate sewers, a scheme in which the control of discharges is achieved by using only sewer overflows. In both cases the sewer overflow works exclusively during wet weather, when either the combined flow in the combined sewer, or the stormwater flow in the separate sewer, exceeds the design limit and the excess flow is directly discharged into the receiving waters.

Figure 2 presents, for both combined and separate sewers, a scheme in which the control of discharges is achieved by using on-line storage basins. For combined sewers, during dry weather, the sanitary flow passes through the on-line storage basin directly towards the treatment plant. During wet weather, the flow exceeding the maximum treatment plant capacity, defined as a multiple of the sanitary flow, accumulates in the storage basin. When the basin is full, the excess flow is discharged into the receiving waters. For separate sewers, the flow enters the storage basin only during wet weather; the stormwater flow in excess of the capacity that can be sent to the treatment plant with the sanitary flow is accumulated in the storage basin. When the basin is full, the excess flow is discharged into the receiving waters.

Figure 3 presents, for both combined and separate sewers, a scheme in which the discharges are controlled by using sewer overflows coupled with off-line storage basins. In both cases, the storage basin works only during wet weather; the flow exceeding the sewer overflow limit accumulates in the basin, until it can be sent to the treatment plant at the end of the storm event. When the basin is full, the flow in excess is discharged into the receiving waters. A by-pass device at the inlet to the storage basin can cut off inflow when it is full, in order to avoid dilution of the more polluted first flush accumulated in the basin.

According to traditional criteria, in Italy, sewer overflow limit is calculated as a multiple (between 2 and 5 times) of the dry weather flow (DWF) in combined sewers, and as a flow between 0.5 and 1.5 l/s, per effective hectare, in storm sewers. It can be easily checked that the application of these two criteria implies the interception of similar stormwater volumes to be sent to the treatment plant.

In the Italian climate, the use of sewer overflows only does not reduce significantly either the number of discharges, or the pollutant loads discharged, or the pollutant concentrations in the discharge. Not even higher sewer overflow limits would assure good benefits, with the negative consequence of high costs for the adjustment of the sewer system and for the operation of the treatment plant [9, 10]. A convenient solution consists in employing sewer overflows coupled with the storage basins, as shown in Figures 2 and 3. In particular, off-line storage basins efficacy is optimal, even with reduced volumes, because of the capture of the "first flush", as pointed out for the observed storm event in Table 3.

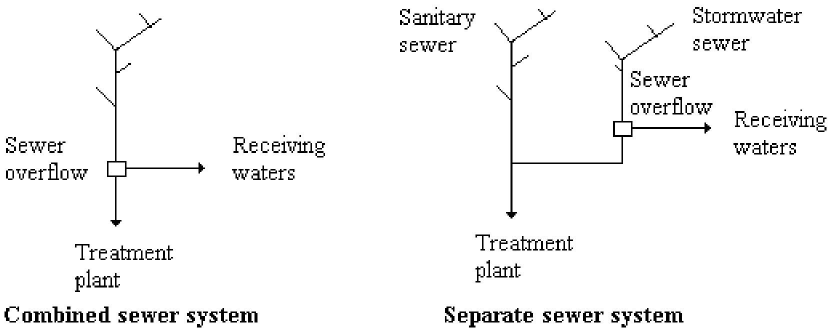

Figure 1. Scheme of combined and separate sewer systems with overflows [11]

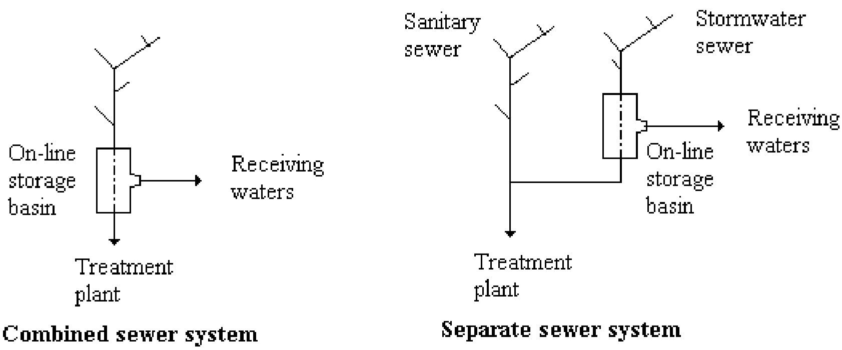

Figure 2. Scheme of combined and separate sewer systems with on-line storage basins [11].

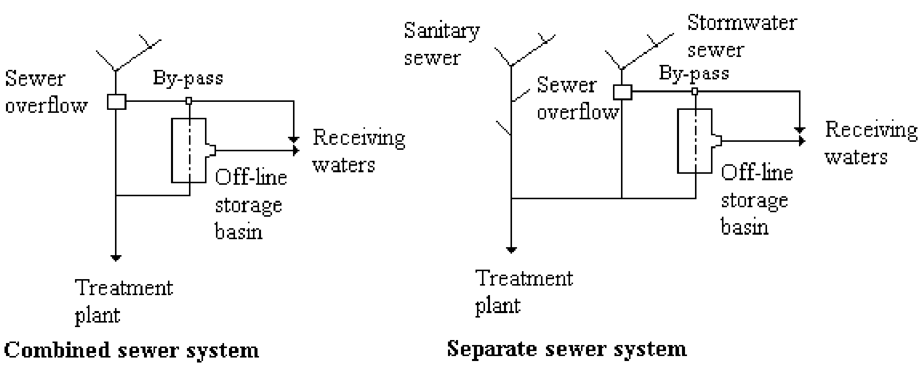

Figure 3. Scheme of combined and separate sewer systems with sewer overflows coupled with off-line storage basins [11].

3. Continuous simulation

Using a global conceptual model [12], which was recently proposed for the estimation of pollutographs during storm events, the efficacy of a sewer overflow coupled with an off-line storage basin in a storm sewer system was tested.

A series of storm events (98), recorded in the urban catchment of Cascina Scala over a period of one year (2000), was simulated for an annual precipitation of 976.4 mm. The variable input parameters included the sewer overflow setting (flow limit), the storage basin volume per effective hectare of the catchment, and a retention constant of the catchment-sewerage system, which is a model parameter that describes the runoff behaviour for the catchment. The efficacy of the system has been expressed as the number of discharges, the volume of water discharged, the suspended solids load discharged and the average and maximum suspended solids concentrations in the discharge.

The model used for the simulation is valid under the hypothesis that the pollutant transport in an urban drainage system during storm events is essentially influenced by convection and that other phenomena like scour, sediment deposition, biological and biochemical degradation of contaminants are negligible. The model is based on the application of the theory of a single linear reservoir system with respect to both quantitative (hydrograph) and qualitative (pollutograph) aspect of runoff. This means that the hydrograph, *q(t)*, is calculated as the convolution integral of the net hyetograph, *p(t)*, and the pollutograph, *y(t)*, as the convolution integral of the pollutant mass rate entering the system, *m(t)*:

$$q(t) = \int_0^t p(\tau)\, h(t-\tau)\, d\tau \tag{1}$$

$$y(t) = \int_0^t m(\tau)\, h(t-\tau)\, d\tau \tag{2}$$

where the term

$$h(t) = \frac{1}{k}\, exp\left(-\frac{t}{k}\right) \tag{3}$$

represents the instantaneous unit hydrograph for a single linear reservoir system, defined as an exponential function of the retention constant *k*. The value for the retention constant is influenced by the physiography of the drainage system and by the precipitation, and it can be estimated from some relationships, which were proposed in several studies [13, 14].

The pollutant mass rate, $m(t)$, which enters the sewer system during the storm event, is evaluated by the model through empirical relationships available in the scientific literature.

The build-up of solids on the catchment surface is simulated using the exponential relationship from the SWMM model of the EPA [15]:

$$m_a = \frac{B}{D}\left[1 - exp\left(- D\, T_d\right)\right] A\ IMP\ \Phi_{imp} + m_r\ exp\left(- D\, T_d\right) \qquad (4)$$

where the symbols have the following meaning:

- m_a is the mass of solids accumulated on the catchment surface [M];
- B is the build-up rate of solids due to the different phenomena which influence the accumulation on the impervious catchment area [M L^{-2} T^{-1}];
- D is a build-up decay coefficient, which represents the dispersion of the particles due to wind, traffic, and biological and biochemical degradation [T^{-1}];
- A is the catchment area [L^2];
- IMP is the ratio of the impervious area to the total area;
- φ_{imp} is the ratio of the contributing impervious area to the total impervious area;
- T_d is the dry weather time elapsed since the last wash-off by rain [T];
- m_r is the mass remaining on the catchment at the end of the previous meteorological event, evaluated from that event characteristics [M].

The surfaces wash-off process assuming that the removed material is proportional to the accumulated mass on the catchment, is simulated through a linear differential equation with a constant coefficient [16]:

$$\frac{dm_a(t)}{dt} = -k_w\ m_a(t) \qquad (5)$$

$$\text{with } k_w = rcoef \left(\frac{rr}{\overline{rr}}\right)^{washpo} \qquad (6)$$

where

- m_a is the mass remaining on the catchment surface at time t [M];
- $rcoef$ is a calibration coefficient [T^{-1}];
- rr is the rainfall intensity over the catchment [L T^{-1}];
- $\overline{rr}$ is a reference rainfall intensity [L T^{-1}];
- $washpo$ is a numerical coefficient, which is usually assumed to be > 1.

The model has been previously calibrated using experimental data from the storm events recorded in the urban catchment of Cascina Scala, listed in Table 1, for which both quantity and quality data are available.

Table 4 presents the values evaluated for the remaining mass m_r at the beginning of each event of calibration, and Table 5 summarises the values adopted in the model calibration of the build-up and wash-off processes.

Table 4. Mass, m_r, remaining on the catchment surface

n. event	1	2	3	4	5	6	7	8
m_r **[kg/ha]**	11.29	45.71	2.27	26.67	14.66	0	2.79	50.97

Table 5. Build-up and wash-off parameter values adopted in model calibration.

	TSS
Surface build-up rate of solids, B (kg/ha/day)	18
Build-up decay coefficient, D (1/day)	0.3
Wash-off calibration coefficient, $rcoeff$ (1/min)	0.2
Calibration coefficient, $washpo$	1.2
Reference runoff rate, $\overline{rr}$ (mm/min)	0.42

4. Results

The simulation results are summarised in the figures reported below. At first, the value of the retention constant k was set as 20 minutes, and the sewer overflow limit, q [l/(s ha)], and the off-line storage basin volume, W [m^3/ha], referring to an effective hectare, were varied and the following data were produced:

- number of discharges into receiving waters (Figure 4);
- water volume discharged into receiving waters (Figure 5);
- suspended solids load discharged into receiving waters (Figure 6);
- average suspended solids concentration in the discharge (Figure 7); and,
- the maximum suspended solids concentration in the discharge (Figure 8).

Then, analogous data were produced changing the value of the retention constant, at first by reducing it to 5 minutes, and then increasing it to 60 minutes. The corresponding results are summarised in Figures 9-13.

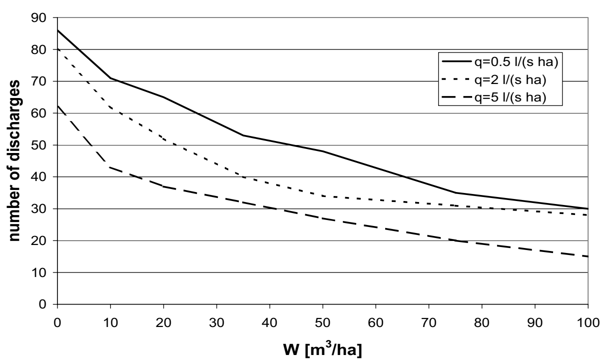

Figure 4. Number of discharges into receiving waters (k = 20 min).

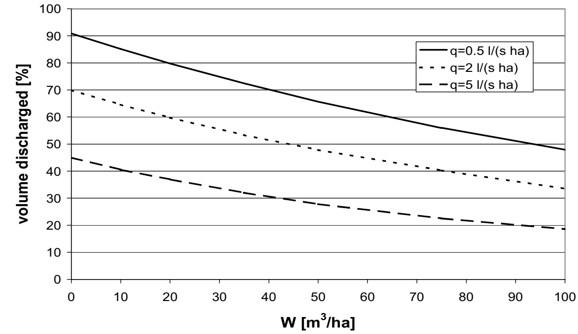

Figure 5. Water volume discharged into receiving waters (k = 20 min).
(annual runoff volume = 8570 m^3/ha)

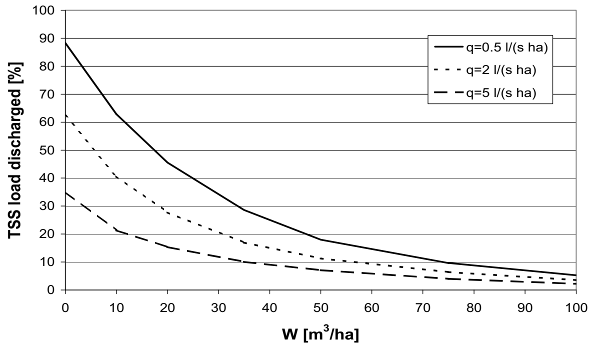

Figure 6. Suspended solids load discharged into receiving waters (k = 20 min) (annual TSS load = 1614 kg/ha)

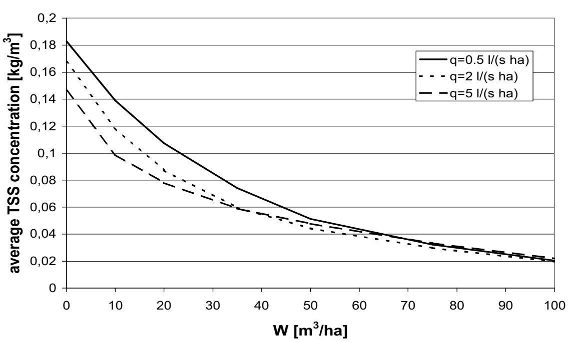

Figure 7. Average suspended solids concentration in the discharge (k = 20 min)

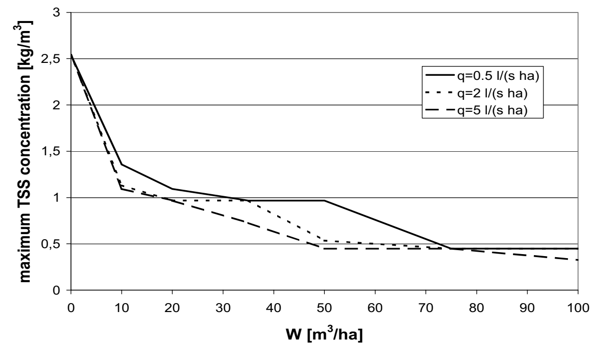

Figure 8. Maximum suspended solids concentration in the discharge (k = 20 min)

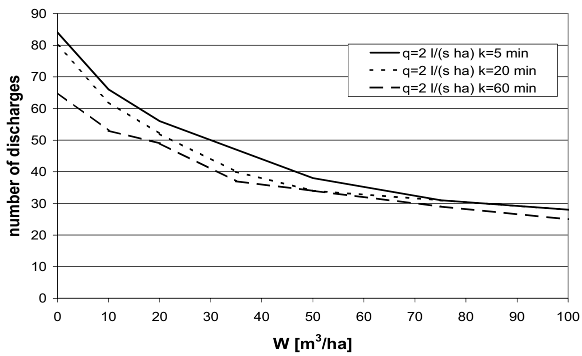

Figure 9. Number of discharges for different values of k (5, 20, 60 min)

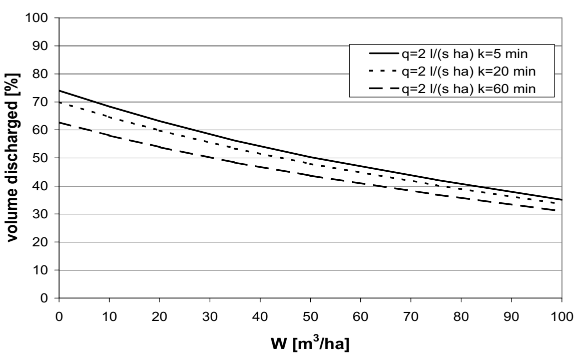

Figure 10. Water volume discharged for different values of k (5, 20, 60 min)

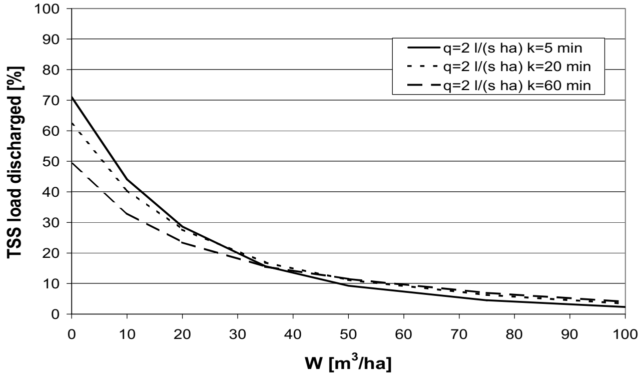

Figure 11. Suspended solids load discharged for different values of k (5, 20, 60 min)

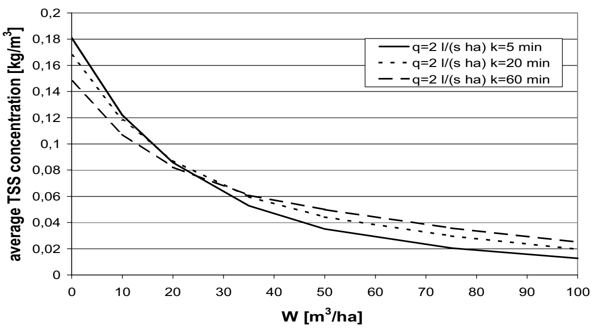

Figure 12. Average suspended solids concentration in the discharge for different values of k (5, 20, 60 min)

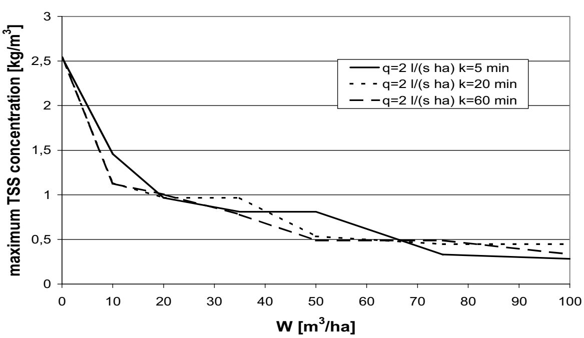

Figure 13. Maximum suspended solids concentration in the discharge for different values of k (5, 20, 60 min)

The simulation results show that, without an off-line storage basin, only the use of sewer overflows with very high flow limits, however incompatible with the capacity of the treatment plant, permits a good reduction of the suspended solids load discharged into receiving waters. Even so the effect on the number of discharges and on the average suspended solids concentration is modest, and the maximum suspended solids concentration is unaffected.

The use of storage basins with normal volumes (40-50 m^3/ha) coupled with sewer overflows allows a significant reduction of both the suspended solids load discharged and their average and maximum concentrations. It can be observed, moreover, that for normal values of storage basin volumes, the sewer overflow limit does not influence much the behaviour of suspended solids load discharged and their concentration, while it still has an effect on the behaviour of the volume discharged and the number of discharges. On the other hand, the use of storage basins with volumes larger than 50 m^3/ha is not recommended, because the achievable benefits are very low. Below 50 m^3/ha, significant reductions of suspended solids load and concentration result from small increases of the storage volume.

At last, it can be easily checked out that the receiving waters control pursued by the use of sewer overflows and storage basins is barely influenced by the variation of the retention constant. Therefore, the hydrological behaviour of the catchment-sewerage system, which depends mostly on the catchment area, does not seem to influence the sewer overflows and storage basins design criteria.

5. Conclusions

This research allowed the estimation of the efficacy of sewer overflows, coupled with off-line storage basins, in the control of negative impacts of urban stormwater runoff on receiving waters.

The number of discharges, their volume and the suspended solids load discharged into receiving waters without treatment were evaluated through continuous simulation over the period of one year, using a global model calibrated with measured quantity and quality data. The results obtained show that the use of sewer overflows only is inadequate, while sewer overflows coupled with storage basins permit high reduction of the pollutant loads discharged into receiving waters, of pollutant concentrations in the discharges, and to a lesser degree, of the number of discharges. It was verified that, in a storm sewer system, the use of a sewer overflow, with a flow limit equal to 1-1.5 l/s per effective hectare, as recommended for Italy, coupled with an off-line storage basin with a volume equal to 50 m^3 per effective hectare, permits a suspended solids load reduction of approximately 80%. In any case, the employment of larger storage basins is not efficient, because the efficacy increase is very small compared to the increase in costs.

The results are obviously influenced by the simplifications of the catchment topography in the model, by the values assigned to the model parameters and by the specific distribution of storm events in the simulated year. In particular, the results obtained for various values of the retention constant, k, could be influenced by the use of a global model and by the assumption of instantaneous mixing, which is more realistic in small catchments, than in larger catchments with a complex urban topology.

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URBAN TREE ROOT SYSTEMS AND TREE SURVIVAL NEAR SEWERS AND OTHER STRUCTURES

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1. Importance of fully-grown greenery for the human environment in the cities

Fully-grown greenery in towns and cities, such as trees and shrubs, serves in many different ways which are important for the society. We are able to distinguish and define the benefits of such greenery from different points of view: dendrologic values, hygienic and ecologic importance, climatic functions, historical values, social and psychological values, etc. More detailed analysis of such questions is beyond the scope of this paper.

All of us recognise the difference between a habitat surrounded by well growing trees and other vegetation, in contrast to depressive mood arising from canyons of empty streets surrounded only by uniform grey concrete walls of panel-houses. That is why it is absolutely necessary to protect the urban greenery against negative impacts of construction activities. However, it is not easy to assure optimal growing conditions for trees situated in narrow streets, when considering safe distances from the engineering network. The problem is getting even more complicated due to full drainage in urban catchment. Urbanisation usually leads to inappropriate dehydration of soils in addition to many others unfavourable hydrological and hydraulic aspects of technical solutions of drainage. The lack of available water in soils for tree root systems results in stimulation of their longitudinal growth in a desperate search for water. Roots are thus forced to penetrate into more distant, mostly non-traditional water sources with consequences of damaged water pipes and sewer networks, regions of evaporative shade, and therefore also beneath pavements and foundations of buildings. Beneficial properties of trees and shrubs can naturally occur only when the urban greenery in the given area exists in sufficient amounts, is sustainable and is in a good functional state (i.e., alive and healthy) – dead trees do not work.

An increasing pressure by specialists implementing wide measures of greenery protection from the viewpoint of urban ecosystems resulted in many positive achievements. E.g., financial evaluation of urban greenery was produced for the Czech Republic by the Agency of Nature and Landscape Protection during recent years, which can be applied in practice by the appropriate specialists. The total price includes real costs associated with planting and a long-term maintenance of trees or shrubs and also their ecological evaluation, which the individual trees represent within the framework of urban ecosystems. This was possible on the basis of

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certification of a raw of intimations related to the act No.279/1997 Sb. "On the evaluation of properties". The estimated economical value of trees thus helps increase the prestige of urban greenery in legal acts in the cadastre of municipalities, especially during the building permit procedures.

Tree maintenance represents an important part of the care for urban greenery. It is a continuous process, whose aim is preservation of the character and the purpose of urban greenery on the cadastre of municipalities. This includes systematic care for trees and shrubs according to their categories and assuring their functions, operation and aesthetic appearance. Maintenance also includes replanting, thinning of stands in the framework of control of their development, removal of natural seeding and eventual sprouts, protection against infections and pests, fixing of damages and mitigation of negative impacts of different activities on the urban greenery. Additional necessary work according to the functional state of the greenery, cleaning and keeping an order is considered here too. For the above-mentioned care is not sufficient to draw tree planting schemes into the corresponding plans or to present statistical documentary evidence. It is necessary to activate the system of public administration. Each tree or planted area falls into one of the 17 classes of maintenance according to its importance and category. This begins with category 1: Lawns, up to the category of Trees in avenues eight years after planting. Minimum and optimum technology of maintenance is elaborated for each class of maintenance. The owner or his executive manager is obliged to regularly maintain all functional items of urban greenery. Care of greenery is stabilised in the City of Brno at present. There are about 260 registered areas of urban greenery and parks in the city of Brno. The area occupied by this greenery cannot be reduced. On the contrary, there is a tendency to reintroduce the greenery also into those parts of the city, where it was destroyed in the past.

2. Urban infrastructure, construction and greenery

Increasing density of human population brings new social and ecological problems. These problems have been usually solved by purely technical means associated with urban infrastructure, such as traffic, water supply, electricity, gas, heating network, and especially urban drainage. Missing or impaired sewage system has the most severe impact on degradation of the urban environment. This is usually discussed in the technical literature only in relation to the receiving water as a part of integrated system of urban drainage. In our opinion, the urban environment includes unavoidably also urban greenery. Here the sewers plays a double role: (1) huge and deep construction of sewerage networks represents one of the major threats to urban greenery, but (2) information needed for their construction can be also used for evaluation of living conditions of greenery. From the technical viewpoint the sewerage network represents a deep-laid inert element which should have long life-span. However, realistic examples found during reconstruction and maintenance of sewerage show that this is not always the case, e.g., in the City of Brno, about 44% of the drainage infrastructure is older than 40 years and does not work properly. This can also be due to adverse effects of urban greenery; its roots often intrude into the sewers, destroying their structure and functions. Greenery may be also harmful to buildings; it often causes desiccation of clay soils and their shrinking which may significantly damage the stability of structures [1]. This means that such problems are serious for a sustainable urban environment and must be solved by a well-coordinated and simultaneous action. Here we want to focus on the possibilities how this can be implemented from the viewpoint of preserving greenery and using modern technology.

The city of Brno published the bylaw No. 10/1994 on the basis of the legislative material for protection of urban greenery. This document must be considered at each level of preparation of projects and especially during the implementation of all construction activities. Observing this bylaw brings investors to the same responsibility to protect trees as any other already built structures and other parts of infrastructure according to the valid Czech National Standard No.73 6005 "Space arrangement of conduit of technical equipment". No materials should be placed close to trees during any construction operations. Tree stems and root systems should not be buried under construction or waste materials or damaged or exposed by lowering the original level of ground surface within the largest area around trees as possible. In case of damage, the builder is obliged to assure immediately proper treatment by a specialist. Only the type of networks determines the minimum distance between engineering networks or their protective structures and trees or shrubs at present. This distance is 2.5 m for heating network, gas pipes and sewers, and 1.5 m for water pipelines, electric and communication cables.

Improvement of the urban ecosystems is not possible only by abiding the official rules or establishing especially protected areas of the most important urban greenery. The principle is that the human environment, which is being created by people, should always be beneficial for the people. Therefore also structures or other technical activities should not harm trees, which are beneficial for people. In contrast, it is not acceptable that a wrongly established new greenery would damage already existing structures or reduce the living comfort, e.g., by permanent shading of apartment houses by overgrown trees, impairment of building foundations by tree roots or their intrusion into sewers or some areas of municipal engineering infrastructure. The main precondition of successful solution of relationships between large urban greenery - trees and shrubs - and structures or urban infrastructure is a background knowledge of properties, natural behaviour and unavoidable needs of this – we should not forget that - living material.

3. Main eco-physiological properties and needs of woody species

Very large plant surfaces, foliage and roots, are most important for metabolism of nutrients and energy. Many close-to-ground-living small herbaceous species almost do not need other vegetative organs (if not considering other, e.g., generative organs as flowers or seeds). Much larger woody species are typical by creating big lignified above-ground skeletons carrying leaves and similar below-ground skeletons anchoring trees in soils and carrying fine roots root systems. Such huge structures allow woody species to occupy large volumes of environment for each individual. They can distribute their leaves and roots at long distances from the original places where they started to grow. This enables their survival under unfavourable conditions.

3.1 TREE STRUCTURE AND FUNCTIONS

3.1.1 Leaves and crowns

Photosynthesis as a process of assimilation of carbon from carbon dioxide and of hydrogen from water requires radiant energy. The whole above-ground part of woody species is determined by an ultimate need to assure optimum needs of intake of radiant energy and exchange of gases (CO_2 and water vapour). Water comes to leaves via a complex of capillary-size conducting elements (vessels and tracheids), which represent the whole conducting system of wood (xylem). Most water evaporates on extremely large inner leaf surfaces and leaves foliage through the stomata; CO_2 enters leaves at the same time in the same way too. Stomata are tiny openings "vents" through leaf outer surfaces, non-visible by eyes (around 0.01mm in diameter), controlled by a cellular mechanism of plants. Density of stomata and their size are important micro-structural parameters typical for different plant species. Their functioning is mostly dependent on light, air temperature and humidity, but their gentle mechanism is sensitive to aggressive factors in the atmosphere, e.g., high concentration of sulphur-oxide, exhaust gases from cars and dust. Polluted atmosphere in towns and cities therefore causes worsening of stomatal control and higher water loses from plants.

The rate of CO_2 assimilation and water loses by plants are also dependent on the macrostructure of tree crowns, i.e., on the total leaf area per individual tree and spatial distribution of foliage. Leaf area is usually 3 to 6 times larger than crown projected area in trees growing in closed stands and even larger in solitary growing trees. Such tree crowns are well illuminated from sides and therefore can be foliated almost to the ground. Shape and form of crowns are mostly dependent on social positions of trees in forest stands and on their positions in relation to surrounding structures in municipalities. Shade stimulates longitudinal growth of stems (tree height) as well as of branches, "growth for light". Excessive longitudinal growth may destabilise trees mechanically so that tall, but weak stems can be easily broken or felled by wind or snow (and therefore can also damage surrounding structures). This often happens after sudden removal of tree protection, under which it was developed in long-term (e.g., after too delayed thinning of dense tree groups, removal of shading old building, etc.). Trees are mechanically more stable when developing on free areas, but their leaves can suffer by excess of light and overheating if they receive additional light reflected from white walls of south-oriented surrounding buildings during hot summers. Such situation jeopardises functional stability of woody species by extra high requirements for soil water needed for transpiration preventing leaf overheating.

3.1.2 Stems

The skeleton of main stem and also of crown and root branches comprises a conducting structure connecting leaves and roots. Water is transported upwards from roots to leaves and other tissues through xylem, and assimilates (about 20% solute of sugars) are transported downwards through phloem. Transport mechanisms in these tissues are different; phloem is typical by a slight positive pressure (associated with the osmotic potential of tissues), while xylem is typical by substantial negative pressure (suction caused by very strong capillary forces in heated leaf cell walls).

Xylem is protected from the outside by all outer tissues; phloem is protected by a layer of inner green bark and outer rough bark with very good temperature insulating (fire protecting) properties.

Almost whole cross-sectional area of stems or branches is usually conducting in young woody species or their young parts. However only the outer part of this area remains conducting in old trees, this is the sapwood in contrast to non-conductive part - the heartwood around stem centres. It is clear from the Hagen-Poiseuill law that flow of a liquid through a capillary depends on the fourth power of its diameter. Therefore species with large vessels (up to 0.3 mm in diameter), although of low vessel density (around 800 cm^{-2}), e.g. oak, ash, elm, have very efficient conducting systems. They include only a single or a few annual rings and thus have shallow sapwood (e.g. 2 to 3 cm), but this is enough to supply sufficient amounts of water for transpiration. Most other species have narrower conducting elements (around 0.1 to 0.05 mm in diameter or less), which they compensate by their high density (around 5,000 to 20,000 cm^{-2}) and involvement of more conductive annual rings (around 30 to 40 or even more), therefore have deep sapwood (e.g. 5 to 15 cm). Conducting systems composed of large conducting elements are very efficient but also very vulnerable (must be renewed each year), while those composed of narrow elements are less efficient but functionally more durable. The elements can be damaged, e.g., by perforation of their walls mechanically (e.g., by fungi or insects), but also by appearance of bubbles (after releasing gases dissolved in water) during freezing and thawing, which takes place usually over winter. Bubbles also appear by air-seeding under very high tensions within xylem (tens of bars) under severe drought. Structure of water pathways is straightforward in most species with large vessels, i.e., water flows from a certain root to a certain branch (or several of them at the corresponding radial segment of stem), situated just above the roots. If a root is cut, most branches above will die. In most species with narrow conducing elements water is conducted in a spiral or zigzag way, so that it is more uniformly distributed over the whole crown. If a certain root is cut, usually no branch will die, but the whole crown is weakened.

Mechanical injury of tree stems (e.g., stretch of the bark caused by moving of heavy machinery or by vandals) is unfortunately very frequent in urban areas. This is most dangerous during the period of rapid growth in late spring and early summer. Destroying "isolating layers" of bark results in gradual deep desiccation of xylem layers, breaking cellulose walls of conducting elements (therefore impairing their conducting abilities) and also infection by bacteria and fungi. Trees are seriously and irreversibly damaged this way. Importance and consequences of each particular injury is modified by its severity and by the structural differences among woody species.

3.1.3 Roots

Roots are organs serving for mechanical stability of woody species (by their anchoring in soils) and for water and nutrient supply from the soils. They can grow above a certain temperature limit and must be supplied by oxygen needed for their own respiration. Big skeletal roots represent concentrated conducting pathways similar to those in stems or branches, while fine roots represent most of absorbing root surfaces. Permeability of absorbing root surfaces is mostly determined by their microstructure, i.e. root hairs (mostly in herbaceous plants) or mycorrhiza (mostly in woody species), which also help with organic nutrition. Water (highly diluted solution of nutrients) is absorbed through the tiny cellulose (micro-fibril) network of cell walls near root tips, if they are not blocked by impermeable layers forcing the water to penetrate through cytoplasm (which can control the flow), eventually through some intercellular openings. Root tips and mycorrhiza are important especially for inflow of selected nutrients, while content of nutrients in water is not much controlled when water is absorbed through openings in suberised roots. The entire volume of soil with nutrients and water, representing a resource pool for a particular plant specimen (the rhizosphere), is given by the macrostructure of root systems. Absorbing root density (root area per soil volume, m^2m^{-3}) determines the rate of rhizosphere exploitation. In practical terms it is given by the area within the reach of horizontally growing roots and by the depth reached by vertically oriented root branches. Values of maximum length of roots and rooting depth of different tree species are presented e.g., by Válek [2].

Woody species tend to keep a certain typical character of their root systems, although this is hardly possible, because roots must adapt substantially to particular growing conditions in contrasting sites (e.g., presence of large stones, roots of other trees, cracks in rocks, walls, etc.). Even usually deeply rooting species (e.g., oaks or pines) develop only shallow but widespread root systems when they grow in absolutely shallow soils (e.g., on rocks) or in absolutely deep, but physiologically shallow soils (limited by high level of underground water table). The projected area of such root systems is then much larger then the projected area of tree crowns. Usually shallow rooting species can develop at least some root branches reaching deeper soil layers (e.g., spruce roots were observed at the depths up to 2 m) in frequently drained soils. In contrast, trees growing, e.g., in the plains of lowland rivers, lakes, etc., where their lower layers are in direct contact with underground water table, develop relatively small root systems, economising assimilates for growth of their above-ground parts. The projected area of such root systems is then smaller then a similar area of crowns. Root systems can be extremely asymmetrical in relations to stem positions. There can be only little amount of short roots around stems (needed for minimum mechanical stability), while prevailing amount of water is transported through a single big coarse root from a very distant location (such roots can be several times longer than the of tree stem).

Size of root systems in municipalities and therefore also the amount of particular available resources is often determined by a limited free soil volume (e.g., in the vicinity of underground structures, engineering networks, roads, etc.). Storage of water and partially nutrients is then lower than needed for tree development. Situation worsens due to impacts of eventual underground works as well as other anthropogenic activities disturbing soil environment. E.g., the level of underground water decreases close to trenches or freshly exposed slopes. Urban greenery can loose water from precipitation, by establishing mounds or ramparts above the original level of soils, or modification of soil surface (e.g., covering it with an impermeable layer of asphalt pavement or compacting soil surface in playgrounds). Gradually decreasing water supply stimulates longitudinal growth of roots. Water must be then absorbed from very distant places, but trees need years or even tens of years to adapt their roots to a desirable extent. They cannot adapt sufficiently fast when environmental conditions change abruptly. Trees will die, if the environmental change is too fast or too great (outside the acceptable physiological limits).

Mechanical damage to tree roots frequently occurs in places close to buildings and other structures. Usually large coarse roots are cut and thus destroyed during earth works. This seriously jeopardises tree mechanical stability (danger of windfalls and damage to parking cars or even injuries of people), but also their physiological stability, when a single or one of the few major roots connecting a tree with major water sources is interrupted. Each injury causes opening of tree tissues to infections, which secondarily can do much more serious damage than the original injury itself. Trees can be infected also due to weakening their tissues by drought, e.g., after decreasing the tissue water potential below –1.2 MPa. Soil becomes physiologically dry, when it receives salt input, which substantially increases the osmotic part of soil water potential. E.g., a several-days long application of road salt (used for road maintenance in winter) at a certain place means many years long if not permanent rejection of this place by urban greenery. Another question concerns root systems of juvenile saplings of woody species used for new planting. Roots are vulnerable to desiccation and mostly will die, if they are manipulated or transported when exposed to the atmosphere (not protected by surrounding soil or by a special technical means). Partially destroyed root systems then cannot supply enough water to otherwise almost undisturbed foliage and whole trees will die due to loss of balance of appropriate proportions of their major structures.

3.2 FUNDAMENTAL LIFE-GIVING RESOURCES NEEDED FOR URBAN GREENERY

Any tree or shrub needs for their well being include certain physical, namely temperature and illumination, conditions and also sufficient amounts of different substances necessary for life: mineral and organic nutrients from soils, oxygen and carbon dioxide from the atmosphere and water. Serious lack of any of these items in the given environment or a substantial change of physical parameters always jeopardises tree survival. This is very important if we consider, that the life-span of trees, planned for tens or even hundreds years, is often limited by a non-planned lack of a fundamental factor, persisting during a critical period for only a few weeks or even days.

A certain minimum physiological range of temperatures globally determines borders of plant distribution in the landscape in general (the northern border in the direction towards Arctic as well as the upper border in high mountains). Similar borders we can discuss also in terms of high temperatures, but here the limits of plant survival are mostly determined by the lack of water. Low temperatures may be important even in our temperate climate during winter, when physiological processes cannot proceed any more below certain limits (e.g., active water absorption from a frozen soil, enzyme activity in cells, etc.). Sudden drops of temperature (e.g., due to late spring or early autumn frosts) are also unfavourable for plants, flushing buds or still growing but non-lignified plant parts are less resistant to frost and can suffer easily. Unfavourable situation is also faced by artificially heated trees (e.g., when growing near hot water pipelines), where the growing period is prolonged unnaturally – they are becoming more susceptible to cold. Light is an energy, which plants absorb and use by photosynthesising mechanisms, through which they create their complex bodies from simple substances. Lack of light can occur, e.g., in very dense stands, in urban areas for example under the shade of tall buildings, in narrow streets, etc. It results in impairment of normal photoperiodic reactions of plants and disturbance of their development.

Situation is usually not critical for carbon dioxide (CO_2). Concentration of this gas in the atmosphere (around 0.03 $\%_{vol.}$) increases globally. The concentration can be up to three times higher in industrial regions, where large amounts of fossil fuels are burned. CO_2 is a rare exception among industrial exhausts, because its rising concentration is mostly favourable for plants (if neglecting possible impacts of slight imbalances in plant metabolism). Also, we usually do not need to be concerned about oxygen supply. Plants produce oxygen, but partially consume it too (although much less than animals). There is enough oxygen for respiration of all organisms in the atmosphere. However, this is not true when considering plant roots. Unfortunately we can often see that tree roots are buried under layers of asphalt or concrete down to tree stems, or soil is compacted around stems, so that oxygen cannot enter it in sufficient amounts. Roots can also suffer by hypoxia, when soils are flooded over long periods of time or when soil air is replaced by a gas (even a non-poisonous one), released from broken underground pipes. Live plant cells then suffocate similarly as animals. Mineral and organic nutrients are utilised by plants for creating their own bodies (synthesis of specific enzymes), proteins, fats, cellulose, etc. New housing developments have been often built on relatively rich soils, so urban greenery does not necessarily suffers by lack of mineral nutrients. More important could be a lack of humus and organic nutrients, when plants are planted on the "dead" soil, e.g., that excavated from deep trenches. However, we can easily supply nutrients by applying small amounts of compost or artificial fertilizers.

The situation is quite different quantitatively, when considering the following most important factor for life – water. Plants need small amounts of water for building their bodies and as medium transporting nutrients to their tissues. However, they need much higher amounts of water for temperature control of their assimilating system - foliage. Photosynthesis, as the most important metabolic process in leaves, can take place only within rather narrow range of temperatures (and also of water content in leaf tissues). Dark leaves exposed to direct solar radiation absorb high amounts of radiant energy, which is mostly converted to heat. The excessive heat is partially removed by convection, but it is mostly lost by evaporation of water from very large leaf surfaces, i.e., by transpiration. Plants use most of water absorbed from soils for this purpose. Amount of water needed for transpiration of an adult tree in Central European conditions reaches tens up to hundreds of litres per day in large trees or shrubs, or tens of cubic meters per growing season. This represents several hundred millimetres of water column on a stand level. Regular supply of plants with such amounts of water can be a technical problem, especially in dry summers, when water is most needed, but generally lacking. Especially drought (even a short one) often results in massive decline of woody species in municipalities as well as in surrounding landscapes.

4. Quantification of spatial needs of urban woody species

Spatial requirements of urban woody species needed for their long-term survival can be specified on the basis of calculation of balance of individual above-mentioned available environmental resources in a particular part of an urban ecosystem. With respect to the amounts of various materials needed, water plays a major role here. Water balance studies usually focus on hydrological problems at the landscape scale (with known overall area and mean soil depth of the region). This approach can be used in urban areas as well. However, we can apply similar water balance calculations also vice versa, i.e., for calculating spatial parameters of particular, spatially very limited and heterogeneous urban sites, needed for survival of trees growing there. We should know usual input data, such as the site evapotranspiration rates and possibilities of water supply. In addition to such input parameters we also need the quantitative biometrical and physiological data on main water consumers, i.e., individual trees (or their small groups), as they actually grow in urban areas.

Estimation of main biometric data for above-ground parts of trees (such as stem diameter, tree height, crown length and its projected area, etc.) is easy and can be performed using the simplest technology available from the time of Aristotle. There are also commercially available optical measuring systems (e.g. Licor, Hemiview, etc.), which can be used for estimation of leaf area index (leaf area above the ground area unit) or leaf distribution. However, it is a quite different situation when we consider directly invisible below-ground parts of trees. Fortunately new instrument technology has been developed most recently, which can solve this problem (Fig. 1).

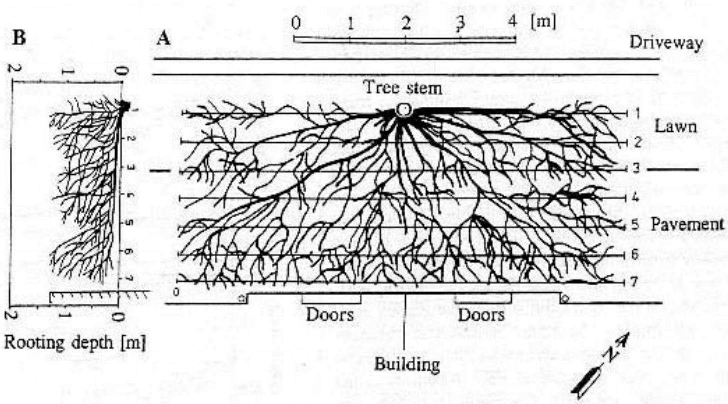

Figure 1.: Example of spatial distribution of coarse roots in the adult maple tree in an urban area obtained when using the GPR (georadar) method. Part of the image under pavement was taken trough the asphalt layer (modified after [3]).

Four methods for studies of root systems of large trees were successfully tested at the Mendel University of Agriculture and Forestry in Brno in cooperation with several other institutions (Cermak and Kucera [4,5], Hruska et al. [6], Nadezhdina and Cermak [7], Cermak et al. [3], Stokes et al. [8], Nadezhdina and Cermak [9], Aubrecht et al. [10]). These methods are: ground penetrating radar (georadar, GPR) that provides 3D images of coarse roots (diameter above 20 mm) from the soil surface down to a depth of several meters. This can even be done through layers of undisturbed materials such as concrete, asphalt or water. Fine roots are too small for applicable electromagnetic wavelengths and cannot be visualized this way, but the total rooted volume of soil can be estimated. The Electric conductometric technique is based on simultaneous measurements of tree tissues and surrounding soil to estimate the total absorbing root surface area per individual trees irrespectively of its detailed distribution. It can be applied in places with no metal materials in the examined soil; therefore it works better in forests and parks. Both above techniques are non-invasive. Root systems can be also excavated by the supersonic air stream (the so-called air-spade). It opens even fine roots causing little damage (root systems exposed under gentle water spray can be covered again with the same technique and trees continue to grow). This technique can also be applied for any no-excavation technology in sensitive places in the city (e.g., laying cables and pipes between tree roots in small yards). There is also a good experience with evaluation of proper functions of tree root systems, which is based on measurements of the radial pattern of sap flow in the conductive wood. This is based on the fact that flow in woody layers of different depth below bark corresponds to roots in different soil depths in most woody species. It is most efficient to combine several of these methods according to particular conditions and tasks.

Sap flow technique also provides quantitative data on transpiration, i.e., water consumption by individual trees. This can be estimated for any tree species and in any terrain and environmental conditions. Such estimates are especially necessary in cases, when similar data already measured at the landscape scale in general hydrological studies (e.g., water consumption of forests in the mountains, lowlands, etc.) cannot be applied to much more specific and variable urban conditions. Using the above technologies we can quantify objectively the biological input data, which are needed for precise water balance calculation for any urban site and woody species occurring there (Fig. 2).

5. Conclusions

We can expect an increasing number of problems for urban greenery associated with reconstruction of linear parts of municipal infrastructure in the nearest future. Especially reconstruction of sewer networks made of inert materials (existing sewers are usually old and of a poor functional state) represents a great risk for fully-grown urban greenery, because such structures are laid deep and require opening of wide areas of the ground surface, often planted with woody species.

Fully-grown urban greenery is an unavoidable part of human environment and should be protected the same way as other systems of local infrastructure. However, recognising the current density of engineering infrastructure networks in streets (not housed in ducts), there is no area left for planting new greenery. After cutting down the existing trees, it is not possible, according to the valid spatial directives, to replant them in the same areas.

Minimal distances of trees from sewers and other installations, such as urban drainage structures and water supply networks, given in the corresponding National Standards (1.5–2.5 m) represent only a simplified and schematic empirical solution of spatial problems. They lack background information about tree root systems and their development, which was not available during their earlier preparation.

New instrumental methods applicable to whole tree root system investigations provide such information, and therefore enable measurements of entire "tree-construction systems" and their interactions. This can be achieved through modified water balance calculations, which are especially important in spatially limited urban locations.

Such objectively determined spatial requirements are needed for assuring sustainable co-existence of urban structures and fully-grown greenery with long life-span (including minimum distances), and should serve for the management of greenery in urban areas.

The minimum life space for urban greenery should be protected legislatively as well as by special technical means (e.g., plastic walls or underground containers) and also by assuring corresponding maintenance operations including regular water control, eventual fertilisation and periodical health control. For example, the City of Brno included in the above-ground tree part monitoring the Viridis system. It should work even better if it is combined with the newest instrumental methods visualizing the underground tree part.

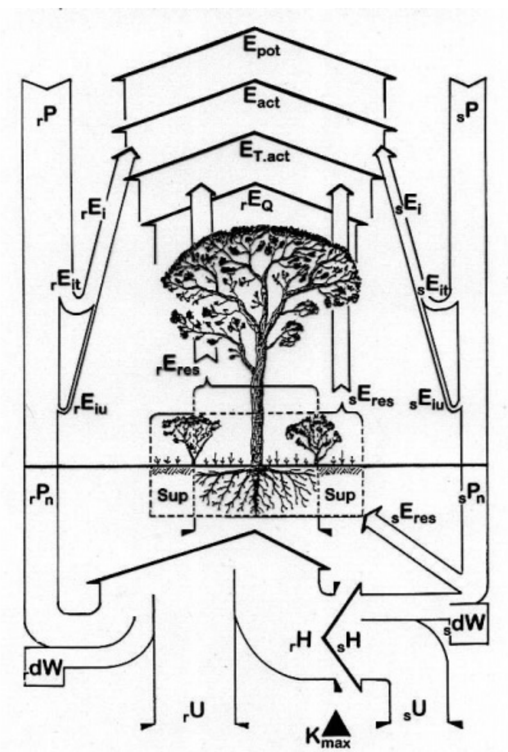

Figure 2. Scheme of components of the detail forest stand water balance, which is also applicable to a single tree scale. Width of the arrows corresponds to the relative water flow rates (per growing season) expressed as percentage of potential evapotranspiration (E_{pot}). Terms without any prefix symbolise values of variables valid for the entire stand, terms with the prefix ***r*** symbolize values valid for rooted volume of

soil and with the prefix ***s*** for the supplementary volume of soil (available around, but not actually rooted). Thus $_{r}P$ and $_{s}P$ is precipitation in the open above the actual rhizosphere and above the supplementary volume of soil, respectively; similarly $_{r}P_n$ and $_{s}P_n$ is net precipitation above the same areas. E_{pot} is potential evaporation (dashed line shows higher value of E_{pot} in dry weather), E_{act} is actual evaporation. $E_{T.act}$ is actual evapotranspiration (evaporation excluding interception), $_{r}P_Q$ is transpiration of the main canopy species (estimated through the sap flow), $_{r}E_{res}$ and $_{s}E_{res}$ is transpiration of undergrowth plant species plus evaporation from the soil surface, the "residual evapotranspiration" for both above mentioned areas. Similarly E_{it} ($_{r}E_{it}$ and $_{s}E_{it}$) and E_{iu} ($_{r}E_{iu}$ and $_{s}E_{iu}$) is interception by the main canopy and by the undergrowth species, $_{r}dW$ and $_{s}dW$ are terms for soil water storage down to the limit of decreased water availability (pda), $_{r}U$ and $_{s}U$ is water from groundwater table. H is water transported horizontally between supplementary volumes of soil and volume of actual rhizosphere (what is subtracted from the supplementary volume of soil, - $_{s}H$, this is added to the rhizosphere, $+_{r}H$) (modified according to [11]).

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CITY OF TORONTO WET WEATHER FLOW MANAGEMENT MASTER PLAN

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1. Introduction

Development and urban growth within the City of Toronto and surrounding regions has resulted in very intense pressures on the ecosystem, and the alteration of the hydrologic cycle and natural environment.

Urban development has two major adverse effects on wet weather flows; increased stormwater runoff quantity and degradation of stormwater runoff quality. The increase in impervious area, changes in surface grading and high surface runoff associated with urban development all act to reduce stormwater infiltration into the ground and increase the volume and peak flow rates of stormwater runoff to receiving waters. Wet weather flow also results in combined sewer overflows (CSOs), storm sewer discharges, and infiltration and inflow (I/I) into the sanitary sewer system causing treatment plant by-passes all of which contribute to degraded water quality in area watercourses and the waterfront. The consequences of these negative impacts include increased flooding and erosion, physical destruction of terrestrial and aquatic habitat, reduced stream base flow, postings of recreational beaches, nutrient enrichment, contaminated sediments, stressed aquatic communities and degradation of the overall environment.

More recently the impacts, summarized through the Metropolitan Toronto and Region Remedial Action Plan Stage 1: Environmental Conditions and Problem Definition report [1], have contributed to Toronto's designation as one of 42 polluted areas of concern within the Great Lakes basin.

In the absence of a legislative requirement for wet weather flow management for existing urban areas, previous wet weather flow control initiatives were driven in large part by the need to address local flooding problems and impacts on recreational beach areas. Although source control options have been considered and implemented to varying degrees, to date, the problems have largely been addressed through the construction of infrastructure and end-of-pipe treatment facilities. Although these initiatives represented significant efforts and provided local environmental improvements, it was recognised that a watershed based strategy was required to provide a more comprehensive and consistent approach to mitigating the impacts of wet weather flows.

J. Marsalek et al. (eds.),
Enhancing Urban Environment by Environmental Upgrading and Restoration, 57-78.
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2. Background

In 1997, the City of Toronto initiated the development of a Wet Weather Flow Management Master Plan (WWFMMP) to address the impacts of wet weather flow. Instead of focusing on site specific wet weather flow issues, the plan development considered the whole natural hydrologic cycle within the context of watershed management and ecosystem protection.

The Master Plan was developed following the planning principles of the Province of Ontario's Environmental Assessment Act incorporating broad public consultation at key decision points. The master planning process was defined by four steps:

Step 1 - data gathering, problem definition and policy formulation;
Step 2 - development of the Wet Weather Flow Management Master Plan;
Step 3 - implementation of the Master Plan; and
Step 4 – ongoing monitoring and updating of the Master Plan.

Step 1 of the Master Planning Process included a synthesis of background information based on previous initiatives and current practices, found both nationally and internationally; identification of wet weather flow (WWF) impacts; the consolidation of existing legislation, policies and guidelines relative to WWF and the preparation of a draft Wet Weather Flow Management Policy Paper.

A new philosophy was adopted for the development of the Master Plan which emphasised control of rainwater where it falls to minimise the amount of stormwater runoff generated from a site. Following the runoff pathways from lot level to receiving waters, a hierarchy of management practices and controls were developed, starting with at source controls (lot level), followed by conveyance system controls, and then end-of-pipe controls.

Step 2 of the master planning process, initiated in 2000, involved developing the Wet Weather Flow Management Master Plan and was completed in 2003. Development of the WWFMMP included establishing targets, filling data gaps, development and assessment of strategies for controlling wet weather flow impacts and preparation of an implementation plan. Concurrent with the development of the WWFMMP, a Wet Weather Flow Management Policy and a funding mechanism to support implementation of the Plan were also developed.

This paper summarises the work undertaken in Step 2 based on the material document in the WWFMMP Technical Reports [2, 3, 4, 5, 6 and 7].

3. Study Area

The Master Plan development focused on the 640 km^2 area contained within the City of Toronto boundaries. However, the study extended to include the six major watersheds of the Rouge River, Highland Creek, Don River, Humber River, Mimico Creek, Etobicoke Creek and the lake-based watersheds draining directly to Lake Ontario representing an area of about 2,100 km^2 (Figure 1). Only one watershed, Highland Creek, is completely contained within the City boundaries, while the remaining five watersheds extend beyond the City's borders. Table 1 summarises the watershed areas and the portion contained within the City boundaries.

Figure 1. Study Area

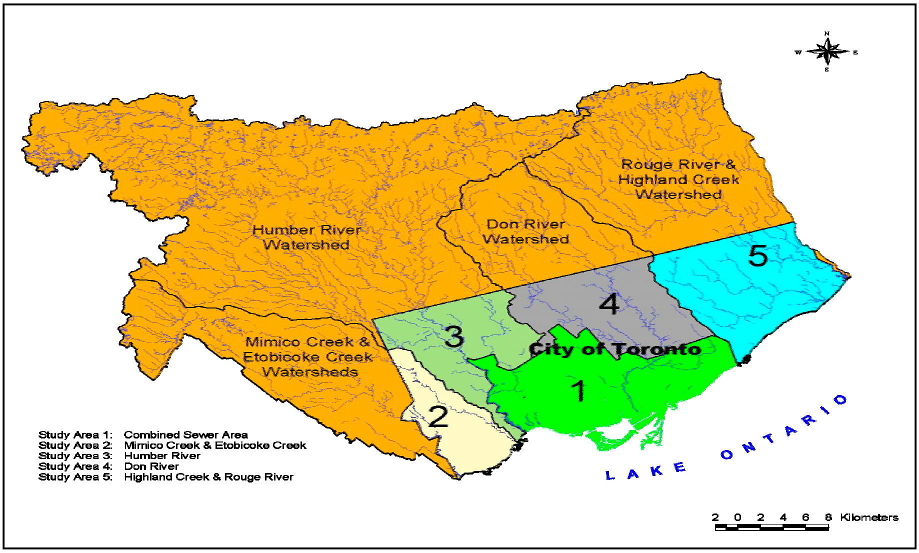


Table 1. Watershed Areas

Watershed	Total Area [km^2]	City of Toronto Portion
Etobicoke	213	10 %
Mimico	83	47 %
Humber	918	16 %
Don	360	51 %
Highland	106	96 %
Rouge	335	11 %
Waterfront	26	100%
Combined Sewer Service Area	171*	100%
TOTAL	2116**	30 %

* Combined Sewer Service Area includes 96 km^2 of the Don and Humber River Watershed, respectively

** excludes ~ 16 km^2 of harbour lands and islands.

By following the ecosystem management approach on a watershed basis [8] the consequence of: urban growth in areas of new development (primarily located in headwater areas of the watersheds); urban intensification within the City of Toronto and other urban centres; together with retrofitting existing urban areas with wet weather flow controls have been analyzed and accounted for in the development of the Master Plan. However, Step 3, the implementation of the Master Plan, is limited to the land area within the City of Toronto and the City will require support from other levels of government and developing partnerships with upstream municipalities, through the Toronto and Region Conservation Authority, to address wet weather flow impacts on the watersheds which flow through the City and then discharge to the waterfront.

4. Study Objectives

With input from the public, a goal, four guiding principles and 13 objectives were developed (presented in Table 2) to direct the development of the Wet Weather Flow Management Master Plan. The objectives, describing more specifically how the goal was to be achieved, were grouped into four major categories: water quality, water quantity, natural areas and wildlife, and sewer system.

Table 2. Study Goal, Principles and Objectives

Goal
To reduce and ultimately eliminate the adverse effects of wet weather flow on the built and natural environment in a timely and sustainable manner, and to achieve a measurable improvement in ecosystem health of the watersheds
Principles
• Rainwater is a resource. As a priority, rainwater (including snowmelt) should be managed where it falls on the lots and streets of our City, particularly before it enters a sewer. • Wet weather flow will be managed on a watershed basis with a natural systems approach being applied to stormwater management as a priority. • A hierarchy of wet weather flow solutions will be implemented – starting with “at source”, then “conveyance”, and finally “end-of-pipe”. • Toronto’s communities need to be made aware of wet weather flow issues and involved in the solutions
Objectives
Water Quality • **Meet guidelines for water and sediment quality**: Contribute to achieving federal, provincial and municipal water and sediment quality objectives and guidelines in area watercourses and along the waterfront. • **Virtually eliminate toxins through pollution prevention**: Contribute to the virtual elimination of toxic contaminants in groundwater and surface water utilising the principle of pollution prevention at source. • **Improve water quality in rivers and the lake for body contact recreation:** Improve water quality for body contact recreation in rivers and recreational areas and reduce posting of beaches by the Medical Officer of Health. • **Improve aesthetics:** Contribute to eliminating objectionable deposits, nuisance algae growth, unnatural colour, turbidity and odour in order to improve the aesthetics of area surface waters.

Water Quantity • **Preserve and re-establish a natural hydrologic cycle**: Contribute to the reestablishment of a more natural hydrologic process to protect and restore groundwater and surface water resources, based on maximising permeability and minimising runoff at source. • **Reduce erosion impacts on habitats and property**: Manage wet weather flows to reduce erosion impacts on stream and riparian habitats on public and private properties and open spaces. • **Eliminate or minimize threats to life and property from flooding**: Eliminate or minimise threat to life and property from flooding.
Natural Areas and Wildlife • **Protect, enhance and restore natural features (e.g., wetlands)**: Contribute to the protection, enhancement and restoration of natural features and functions such as wetlands and riparian and other ecological corridors. • **Achieve healthy aquatic communities**: Contribute to achieving healthy aquatic communities, including warm water or coldwater fisheries as appropriate. • **Reduce fish contamination**: Contribute to reducing fish consumption advisories due to local wet weather sources.
Sewer System • **Eliminate discharges of sanitary sewage**: Eliminate discharges of sanitary sewage including those associated with Combined Sewer Outfalls (CSOs), Storm Sewer Outfalls (SSOs), treatment plant bypasses, illegal cross-connections and spills. • **Reduce infiltration and inflow to sanitary sewers**: Reduce sanitary sewer infiltration and inflows to City design standards. • **Reduce basement flooding**: Manage wet weather flow to reduce basement flooding.

5. Target Setting

Indicators and corresponding targets were developed for each of the objectives to provide a basis for evaluating the effectiveness of alternative wet weather flow management strategies in achieving the stated goal and objectives. Three levels of targets: status quo, moderate enhanced and significant enhanced, to reflect increasing levels of ecological enhancement, were established.

Status Quo (or short term targets): Status quo targets represented existing conditions and ensured no further deterioration in the watersheds. This may require the implementation of measures to mitigate against the impacts due to intensification within the City and/or future development stresses from upstream municipalities.

Moderate Enhancement (or medium term targets): Moderate enhancement targets represent “improved” conditions within the watersheds and subwatersheds within the City whereby: nutrient levels would be moderate and there would be fewer occasions when toxics are present in detectable amounts; conditions would support a more sensitive aquatic community; in non-beach areas, conditions would meet the Provincial Water Quality Objective for swimming beaches 50% of the time; stream baseflow would be higher than existing conditions; wet weather flows would peak at lower levels; stream banks would be more stable; 50% of basement flooding occurrences and contaminated dry weather sources would be eliminated; and the provincial requirements for the control of combined sewer overflow would be achieved.

Significant Enhancement (or long term targets): Significant enhancement targets represented achievement of Provincial Water Quality Objectives and other regulatory requirements; dry weather flow and runoff conditions would be close to historic conditions, resulting in stable stream conditions; fish community objectives in the fisheries management plans would be achieved; basement flooding and contaminated dry weather sources would be virtually eliminated and the provincial requirements for the control of combined sewer overflow would be achieved.

6. Public Consultation

A 24 member Steering Committee was formed to guide the development of the Master Plan. The Steering Committee included 2 City Councillors, 6 City staff from the affected departments, 4 external agency staff and 12 public representatives.

The City used a combination of public consultation methods and devoted significant resources to the public consultation efforts to fulfil the requirements of the Master Planning process under the Environmental Assessment Legislation of the Province of Ontario. While the contribution and effort put forward by the public members of the Steering Committee was significant, extensive opportunities for the broader public to participate was provided through multiple avenues: workshops, public meetings, focus groups, e-consultation, a formal report review period, etc. These opportunities were promoted through various means including direct invitations to the extensive project mailing list, personal contact with community groups and leaders, and newspaper advertisements.

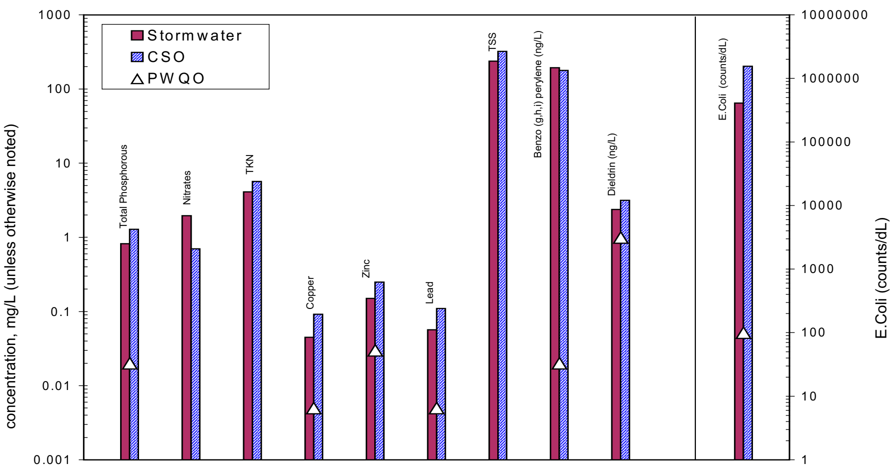
Public and stakeholder advice and feedback at major decision points in the process included: defining existing conditions in Toronto's watersheds; developing guiding principles; defining study objectives; establishing indicators and targets; reviewing a long list of potential stormwater management options; developing evaluation criteria to be applied in selecting the preferred strategy; developing strategies; selection of a preferred strategy and establishing the 25 year implementation plan [9].

7. Determine Existing Conditions

Existing conditions with respect to flow and environmental degradation within area watercourses and along the waterfront were established through a review of previous investigations and studies, collection of field information and data and by applying the calibrated computer simulation models.

Urbanisation within the City as the watersheds were progressively converted from rural land uses has resulted in flooding, erosion, degradation of aquatic and terrestrial habitats, reductions in the quality and extent of biological communities and health risks in body contact recreation areas. This phenomena is typical of all urban areas [8]. In general, the watercourses are degraded, requiring extensive work to restore them to acceptable levels. While past actions directed at improving water quality conditions were focussed on combined sewer overflow discharges, stormwater discharges in urban centres have been found equally problematic. In the past water quality impacts were usually associated with the discharge of combined sewer overflows, which because of the sanitary sewage component, were significant sources of the bacteria, nutrients and other conventional contaminants. However, several studies [10 and 11] have demonstrated that storm sewer discharges are also significant pollution sources. A summary of the event mean concentrations in storm sewer and combined sewer overflow discharges, collected within the City of Toronto, were used to define existing conditions and later used for modelling purposes in the development of the plan. Representative water quality constituents are presented in Fig. 2 and compared to Provincial Water Quality Objectives where they exist.

Figure 2. Constituent Concentrations in Storm Sewer and Combined Sewer Overflow Discharges



In Figure 2, pollutant concentrations in storm sewer discharges are comparable to CSO discharges for most parameters. Furthermore water quality constituents in most cases, are well above both Ontario's Provincial Water Quality Objectives and the Canadian Council of Resource and Environmental Ministers Guidelines.

8. Control Measures

Over 90 potential stormwater management and combined sewer overflow control measures were compiled for consideration in the development of the alternative strategies [12 and 13]. They were categorised as source controls, conveyance controls, end-of-pipe controls, maintenance and operational practices, and special measures which provide a watershed or stream improvement, such as the removal of fish barriers.

9. Computer simulation models set-up, calibration and validation

The setup, calibration and validation of hydrologic/hydraulic and water quality computer simulation models was implemented for each of the six watersheds, the combined sewer service area and the waterfront of Lake Ontario, respectively. The computer simulation models were developed to determine the impact of storm sewer and combined sewer overflow discharges. The models, once calibrated, were used to determine existing conditions and assess the effectiveness of various wet weather flow control options and alternative strategies.

The U.S. EPA Hydrologic Simulation Model HSP-F was used to model wet weather flow impacts within the six watersheds [3-6, & 14], while the combined sewer area was modelled using the Dorsch QQS program [2 & 15]. The Rand two-dimensional hydrodynamic and water quality simulation model [16] and subsequently the Danish Hydraulic Institute Mike – 3D model [17] were used to assess water quality impacts along the nearshore area of the City of Toronto Waterfront [7].

Considerable effort was required to model the hydrological effects of source (lot level) and conveyance controls. For example, there are a number of ways by which rainwater falling on a roof area or the lawn of an individual lot, can reach a stream. Rainwater on a lawn may infiltrate, and later released as baseflow or may infiltrate to the building foundation drains and then flow directly to the storm, sanitary or combined sewer system. Rainwater on a roof will flow through the roof downspouts and then directly to the storm, sanitary or combined sewer systems, or if disconnected from the sewer system, flow onto the lot area, ideally pervious, where it can infiltrate. Industry experts in North America and Europe were consulted on how best to incorporate these various pathways into the models. However, the level of detail proposed had not been previously modelled in similar studies. The computer simulation models were setup to represent the physical integration of lot level source controls with conveyance and end-of-pipe controls

An analysis of 20 years of historical rainfall data collected from within the City of Toronto was undertaken to select a typical rainfall year based on event frequency, annual and seasonal volumes, average and peak event volumes and average and peak antecedent period [6]. The rainfall record for 1991 was used as the basis for modelling work related to assessing the impacts of the various strategies.

The HSP-F model was calibrated for flow characteristics and water quality constituents based on Water Survey of Canada stream gauging station data for the period 1991 to 1996 and an extensive water quality monitoring program undertaken in the 1990’s [18, 19, 20 and 21].

The quantity calibration for the watersheds concentrated on both overall volumes of water discharged annually (Table 3), and the flow frequency distribution. A representative plot of the flow frequency for one watershed (Mimico Creek) is presented in Figure 3. Overall, there is excellent agreement with the total annual volumes discharged, and the flow frequency distribution.

Figure 3. HSP-F Model Calibration for Mimico Creek Flow Frequency (1991 – 1994)

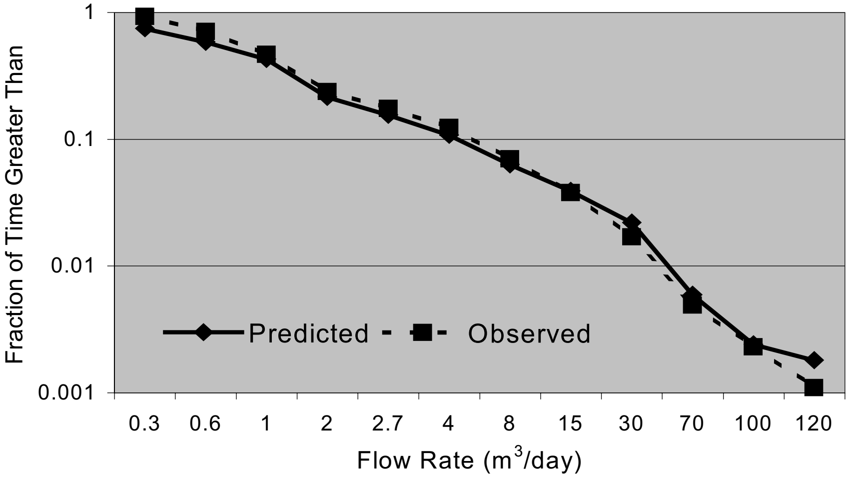


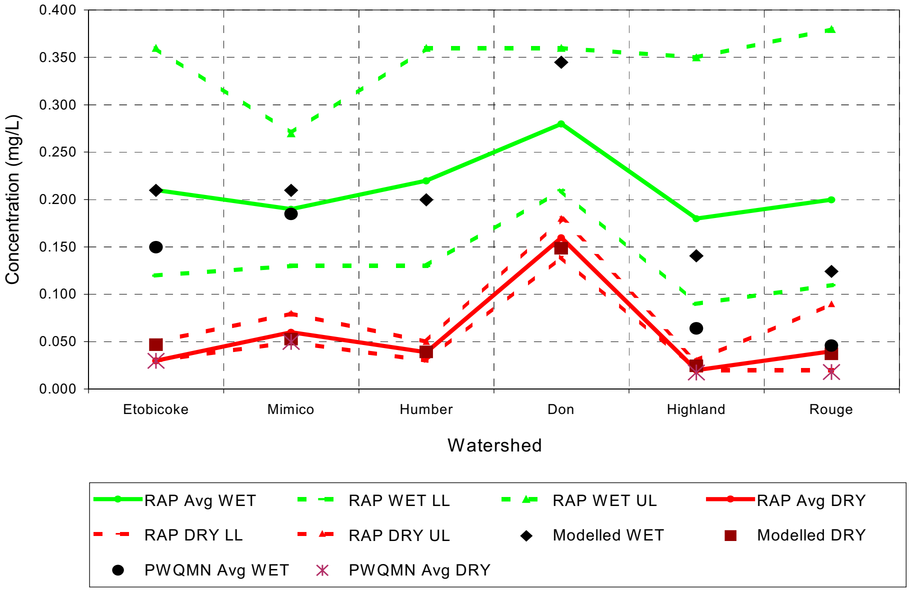
Table 3. Summary of Water Quantity Balance using HSPF (volumes in m^3 $x10^6$)

	Mimico		Humber		Don		Highland	
	Target	Model Results	Targe t	Model Results	Targe t	Model Results	Target	Model Results
Calibration Period Total Volume (1994-1996)	32.6	33.1	64.8	59.6	124.1	124.8	77.7	86.4
Validation Period Total Volume (1991-1993)	51.8	51.6	73.5	71.6	136.4	140.3	139.0	133.9
Typical Year Total Volume 1991	14.2	14.6	23.7	21.7	130.4	136.7	41.1	38.0

The water quality model was calibrated with observed concentration measurements that included complete wet weather events and dry weather inter-event periods. Parameters selected as representative of water quality concerns in streams and the waterfront included: total phosphorus, total suspended solids, total kjeldahl nitrogen, nitrate plus nitrite, E. Coli bacteria, total lead, total copper, total zinc, dieldrin, and benzo(G,H,I) perilene, and temperature.

A representative comparison of model predictions to measured instream concentrations is presented in Figure 4 for total phosphorus. The dashed lines represent the 95% confidence interval for mean wet weather and dry weather concentrations, represented by solid lines, respectively [20]. Model predictions match under dry weather conditions and are in good agreement with measurements for wet weather.

Figure 4. Comparison of Water Quality Predictions to Measured Data by Watershed – Total Phosphorus



The Dorsch QQS model calibration was undertaken using sewer flow data collected by the City of Toronto and event mean concentrations for representative City storm sewer and combined sewer overflow discharges [18 and 19].

The RAND model was calibrated using current meter and water quality data collected along the waterfront in 1993 [7] and was used to determine present conditions along the waterfront, using inputs to the waterfront (flow and water quality) derived from the HSP-F and Dorsch QQS models.

10. Development of Strategies

Meeting the WWFMMP Objectives requires improvement in both water quality and water quantity conditions in area surface waters, through improved stormwater management. Through a strategic planning process a set of five alternative strategies for the management of wet weather flow impacts were formulated. Each of the strategies were developed, in accordance with the principle of using a hierarchical approach to stormwater management (noted in Table 2), and varied in terms of the target level of improvement expected; levels of source, conveyance and end-of-pipe controls proposed; and level of effort placed in controlling dry weather pollution sources. The set of assumptions used to develop the strategies, across each watershed area, are summarized in Table 4 and described in the following:

Strategy 1: (Maintain Status Quo) involved the implementation of stormwater best management practices (BMPs) that maintain existing environmental conditions with future intensification and additional suburban growth.

Strategy 2: (Opportunistic) involved the implementation of opportunistic BMPs (i.e. those BMPs that could be implemented as opportunities arise).

Strategy 3: (Strive for Moderate Targets - end of pipe focus) involved the implementation of BMPs that strive towards achieving moderate environmental improvements based on a voluntary uptake for source controls and more emphasis on end-of-pipe controls.

Strategy 4: (Strive for Moderate Targets – source control forms) involved the implementation of BMPs that strive towards achieving the same level of moderate environmental improvements as strategy 3 but with the focus on an enhanced level of uptake for source controls and less focus on end-of-pipe controls. Strategies 3 and 4 helped to illustrate the differences in environmental impacts and types of BMPs required when "Voluntary" versus "Enhanced" uptake for source controls are applied.

Strategy 5: (Strive for Enhanced Targets) involved the implementation of enhanced levels of source, conveyance and end-of-pipe control measures that strive towards achieving significant environmental improvements, such as achieving Provincial Water Quality Objectives. Three variations of strategy 5 (5, 5a and 5b) were considered for the Combined Sewer Service Area consisting of different levels of sewer separation.

Table 4. Elements of the Alternative Strategies

Strategy Number	Target Level	Source Controls	Conveyance Controls	End of Pipe	Dry Weather Flow
1	Existing	As required on-site for infill	As required on-site for infill	Existing	Existing
2	Not specific	Voluntary	Time limited exfiltration & filtration	Opportunistic green infrastructure	Limited improvement (based on search & destroy program)
3	Moderate Enhancement	Voluntary	Time limited exfiltration & filtration	Opportunistic & aggressive facilities	Limited improvement
4	Moderate Enhancement	Enhanced	Enhanced exfiltration & filtration (not time limited)	Opportunistic green infrastructure	Extensive improvement (based on long term infrastructure replacement)
5	Significant Enhancement	Enhanced	Enhanced	Opportunistic & aggressive facilities	Extensive improvement

Consistent with the requirements of the Environmental Assessment Act the alternative strategies were assessed using evaluation criteria that incorporated the effectiveness in meeting the objectives of the WWFMMP as well as social/cultural, environmental and economic factors. A 25 year time frame was used to establish cost estimates for all strategies.

11. Water Quantity and Quality Response for Alternative Strategies

The water quantity and quality response to the alternative strategies were determined within the mainstem of each watercourse, their tributaries and along the waterfront.

The most significant improvements were projected within the Highland Creek watershed because it is wholly contained within the City of Toronto. Correspondingly, the smallest changes were projected in the Humber River, Etobicoke Creek and Rouge River where only 10 – 16% of their watershed is contained within the City's boundaries.

The change in baseflow as a percentage of mean annual flow provided an indication of how well the whole hydrological cycle was balanced (i.e. to reduce the amount of peak runoff and increase the amount of baseflow). Baseflow, which is the amount of flow in the watercourse during dry weather, is the result of precipitation infiltrating into the soil and recharging the groundwater: the main source of water in these streams. Mean annual flow is a reflection of the average volume of flow in the stream during wet weather. The overall was to increase the percentage of baseflow to mean annual flow.

Tables 5a, 5b summarise the ratio of baseflow to mean annual flow as a percentage, at the mouth of each watercourse and a typical tributary contained wholly within the City for each of the five alternative strategies, respectively.

The change in the ratio of baseflow to mean annual flow varies among the watersheds for each strategy reflecting the effect of the percentage of the watershed contained within the City, and thereby affected by the controls proposed, and the varying soil conditions which affects the amount of stormwater infiltration.

Table 5a. Comparison of Baseflow to Mean Annual Flow Response in the Mainstem of Each Watershed (%)

Strategy	Rouge	Highland	Don	Humber	Mimico	Etobicoke
Status Quo	49.8	34.7	53.0	41.2	39.0	45.0
Strategy 2	50.3	41.9	55.0	35.0	44.0	65.0
Strategy 3	50.3	48.2	55.0	42.0	44.0	64.0
Strategy 4	50.8	49.4	56.0	42.7	54.0	71.0
Strategy 5	50.8	47.0	57.0	42.6	64.0	71.0

Table 5b. Comparison of Baseflow to Mean Annual Flow Response in a Representative Tributary for Each Watershed (%)

Strategy	Rouge	Highland	Don	Humber	Mimico	Etobicoke
Status Quo	49.0	31.5	38.0	34.2	56.0	N/A
Strategy 2	52.0	38.8	48.0	37.1	62.0	N/A
Strategy 3	52.0	38.8	49.0	37.3	62.0	N/A
Strategy 4	55.0	46.9	60.0	41.5	68.0	N/A
Strategy 5	55.1	46.8	60.0	41.6	68.0	N/A

The water quality response to the five alternative strategies is represented in Tables 6a, 6b and 7 which contain a summary of predicted concentrations for total phosphorus at the mouth of each watershed and E. Coli bacteria levels at the waterfront, respectively.

Table 6a. Comparison of Average Wet Weather Total Phosphorus Response in The Mainstem of Each Watershed (mg/L)

Strategy	Rouge	Highland	Don	Humber	Mimico	Etobicoke
Status Quo	0.126	0.155	0.27	0.378	0.158	0.101
Strategy 2	0.122	0.138	0.15	0.339	0.140	0.077
Strategy 3	0.113	0.086	0.12	0.327	0.134	0.077
Strategy 4	0.112	0.061	0.12	0.316	0.122	0.072
Strategy 5	0.107	0.011	0.095	0.29	0.091	0.072

Table 6b. Comparison of Average Dry Weather Total Phosphorus Response in the Mainstem of Each Watershed (mg/L)

Strategy	Rouge	Highland	Don	Humber	Mimico	Etobicoke
Status Quo	0.038	0.022	0.190	0.058	0.077	0.044
Strategy 2	0.035	0.019	0.073	0.057	0.056	0.032
Strategy 3	0.034	0.015	0.050	0 .057	0.056	0.032
Strategy 4	0.034	0.006	0.051	0.059	0.028	0.025
Strategy 5	0.032	0.002	0.025	0.056	0.027	0.025

Table 7. Predicted response for E. Coli levels along Waterfront Beach Areas (numbers of hours in exceedence of 100 counts/100 ml) [simulation period = 2928 hours]

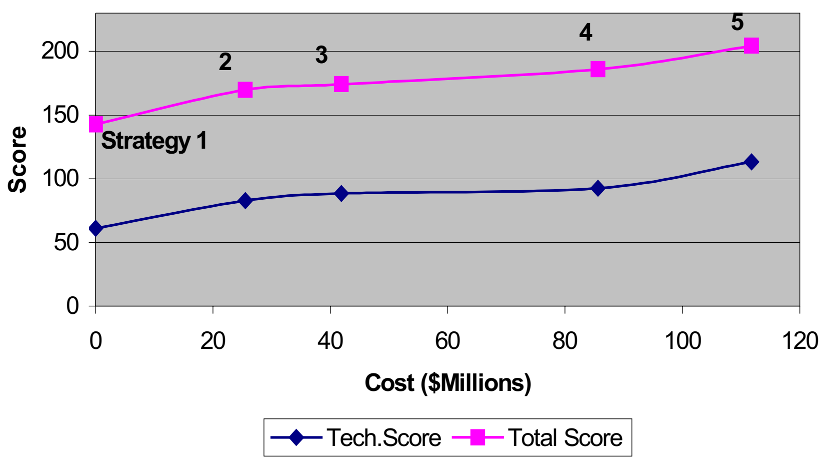
Location	Existing	Status Quo	Strategy 2	Strategy 3	Strategy 4	Strategy 5
Rouge Beach	581	697	661	648	638	623
Bluffers Park Beach	48	48	0	29	0	0
Scarborough Bluffs Park	414	414	225	306	196	0
Eastern Beaches	639	636	176	392	58	0
Ashbridges Bay	2892	2891	2765	2874	2665	277

Cherry Beach	604	660	355	306	386	0
Mid Harbour	2404	2416	1321	1373	1453	150
Center Island Beach	0	0	0	0	0	0
Hanlon Beach	0	0	0	0	0	0
Gzowski Beach	1289	1298	986	890	694	590
Humber Bay Shores Park	681	707	561	427	313	142
Amos Waite	515	478	309	272	195	131
Prince of Wales	424	320	228	160	203	96
Long Branch	641	503	408	390	382	355
Marie Curtis Beach	711	559	489	491	464	453

12. Feedback Received on the Strategies

A cost-effectiveness relationship for a typical subwatershed is presented in Figure 5**.** The graph presents an assessment of each strategy in terms of cost and effectiveness in achieving the study objectives, based on technical scores (e.g. achievement of water quality objectives) and total scores (based on evaluation criteria), respectively. In general, no single strategy stands out as the optimum and nor is a"kink" in the curve observed where when could identify a point of diminishing improvements as more funds are invested. Rather, the curve shows a relationship where environmental quality continues to improve as more funds are invested.

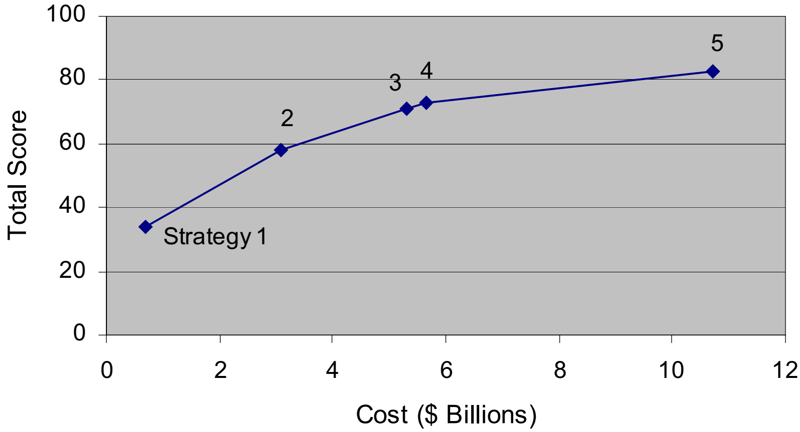
Figure 5. Cost Effectiveness of Strategies for a Representative Subwatershed



However, a cost-effectiveness relationship for the central waterfront, Figure 6**,** similar to that presented in Figure 5 for a sub-watershed, has a concave shape. This indicates that diminishing environmental improvements are achieved as more funds are invested. However, the curve does not show a definitive 'kink" point to guide the selection of a preferred strategy. The difference in the shape of the curves in Figures 5 and 6, can be attributed to the fact that waterfront water quality is significantly improved in comparison to the subwatershed areas.

Based on these curves, it was concluded that the cost-effectiveness curves for the strategies did not provide a definitive basis for selecting a preferred strategy, from among the five strategies. In addition, the total score based on an evaluation of social/cultural and other environmental and economic factors indicated that the strategies did not have a significant negative impact that would preclude selection of any of the strategies.

Figure 6. Cost effectiveness of Strategies for Central Waterfront Sector



13. Preferred Strategy

Consistent with the requirements of the Environmental Assessment Act, the alternative strategies were assessed using the established evaluation criteria. Throughout the Plan development process, feedback from the steering committee and the broad public consultation process stressed that the WWFMMP should strive to meet Provincial Water Quality Objectives [9].

Based on the study philosophy and hierarchical principle, public feedback and the results of the evaluation process, Strategy 5 was selected as the preferred wet weather flow management strategy.

In the combined sewer service area, three versions of strategy 5 (5, 5a and 5b) were assessed, each consisting of a different level of sewer separation. Strategy 5 consisted of complete sewer separation (road sewer separation as well as sewer separation on private property). Strategy 5a included road sewer separation as a measure to help eliminate basement flooding. In the remaining parts of the combined sewer service area, road sewer separation would be implemented on an opportunistic basis where soil conditions permit. The combined sewer overflows that remain, after accounting for the benefits of the proposed source controls and conveyance controls, would be managed through the implementation of underground storage facilities to provide detention, followed by treatment to meet the requirements of the Ontario Ministry of the Environment guidelines prescribed in Procedure F-5-5 for the control of CSO. Strategy 5a did not consider road sewer separation as a measure to manage CSO because, although CSO's would be reduced, storm sewer discharges would increase resulting in little or no improvements to the environment. Strategy 5b consisted of complete road sewer separations but no sewer separation on private property.

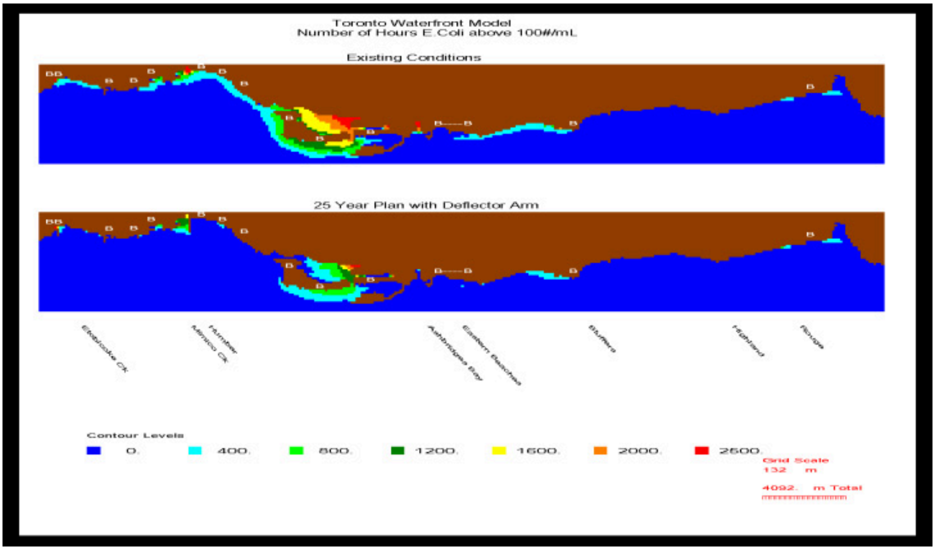
Strategy 5a was selected as the preferred strategy as it was the most flexible in ultimately achieving the study objectives, achieved higher levels of improvements in a shorter timeframe at considerably lower cost and with less social impacts than Strategies 5 or 5b.

To implement the combination of all enhanced source, conveyance and end-of-pipe control measures of the preferred strategy on a City-wide basis and to further achieve the ambitious goal of meeting Provincial Water Quality Objectives in area surface waters, the full Plan implementation may take 75 to 100 years at a cost estimated to be in the order of $12 billion.

Consistent with the planning horizon for Master Plans, a 25 year implementation plan was established, directed at achieving the City's corporate priorities of health and safety (i.e. eliminate basement flooding and provide swimmable water quality at waterfront beaches), infrastructure protection (i.e. prevent stream erosion) and renewal (i.e. eliminate dry weather discharges), intensification (i.e. accommodate growth projected in the City's Official Plan) and the legislative requirements to eliminate combined sewer overflows (i.e. satisfy the Ministry of the Environment Procedure F-5-5).

As an example of the improvements expected in implementing the 25 year Master Plan, Figure 7 presents a comparison of model results between existing conditions and after the implementation of the 25 year plan for the number of hours in which E. Coli levels exceed the 100 counts/100 mL target for body contact recreation.

Figure 7. Comparison of Waterfront E. Coli Predictions for Existing Conditions and the 25 Year Master Plan



The Master Plan also addresses the objectives of the City's Environmental Plan regarding water quality improvements and advances the water quality improvement objectives of the Toronto and Region Remedial Action Plan.

Overall benefits projected through the implementation of the Master Plan are substantial across the City and include:

- swimmable waterfront beaches;
- elimination of combined sewer overflows - in compliance with Ministry of the Environment requirements;
- elimination of dry weather discharges;
- basement flooding protection;
- protection of City's infrastructure from stream erosion;
- restoration of degraded local streams and improved stream water quality;
- reduction of algae growth along the waterfront and in streams; and
- restoration of aquatic habitat.

14. Master Plan Overview

In support of the Master Plan, an implementation schedule [22] was developed prioritising the list of projects to be implemented over the next 25 years, with an overall objective of striving to meet the moderate level targets. The implementation schedule details the measures, individual projects and site locations, together with their projected implementation capital and operational costs in five year intervals. The Plan will be reviewed and its effectiveness in achieving the targets established will be assessed on a 5 year cycle allowing for adjustments to the Plan, if necessary. Regular updating will also permit a review of new and emerging technologies for their applicability and incorporation into the Plan.

The measures contained within the Master Plan consist of: an enhanced public education and community outreach program, enhanced municipal operations including a dry weather discharge remediation program, shoreline management, source controls, conveyance controls, end-of-pipe controls, basement flooding protection works, stream restoration works and environmental monitoring and Plan review. A summary of the proposed works with their associated costs is provided in Table 8.

Table 8. Twenty-five Year Master Plan Summary

Component of the Plan	Capital Cost ($ million)
PUBLIC EDUCATION - city wide over 25 years • Focussed on increasing public awareness	30
SOURCE CONTROLS - city wide over 25 years • Existing ~ 10-15% participation rate • Target of 40% participation rate is proposed	112
MUNICIPAL OPERATIONS - city wide over 25 years • Search & eliminate Dry Weather discharges • Enhanced street sweeping and catchbasin cleaning • Monitoring of Plan implementation and effectiveness	8
BASEMENT FLOODING – emphasis in first 5 years • Focused on cluster areas previously identified • Sewer system upgrading & “home isolation” program	55
SHORELINE MANAGEMENT – implemented in first 10 years • Humber River and Etobicoke Creek deflector structures • Restoration of Highland Creek and Rouge Park Marshes	42
STREAM RESTORATION – emphasis in first 15 years • Focus on protecting City’s infrastructure • Restore aquatic stream habitats • 104 km of stream restoration proposed	131

END-OF-PIPE CONTROLS- implemented over 25 years Green End-of-Pipe: Stormwater Ponds, Constructed Wetlands • opportunistic basis where sufficient open space available • 180 facilities proposed Underground Storage: space limited considerations • necessary to address combined sewer overflows • 16 CSO facilities proposed • 50 stormwater facilities proposed • 4 CSO treatment facilities proposed	 105 491
Total Capital Cost	1,047
Operation & Maintenance Cost (associated the new stormwater control measure)	233

The total capital cost for the Master Plan over the 25 year implementation period is estimated at $1.047 billion resulting in an average annual expenditure of approximately $42 million. The implementation of these new capital works will generate operational and maintenance costs estimated to be $233 million with an accrued annual operating cost of about $16 million projected by the end of the 25 year plan.

To support an expenditure of $1 billion dollars over the next 25 years, various funding options including increasing water rates, levying property taxes, implementing user charges apportioned to the percent impervious area of an individual lot, development charges and grants/subsidies, etc., are being assessed as potential sources of revenue to finance the Plan. The financial impact on the average home owner has been estimated to be in the range of $30 to $70 per year depending on the funding option.

A Wet Weather Flow Management Policy [23] was also developed to support the implementation of the Plan. The Policy, developed with input from the public, internal City staff across the affected departments and the appropriate government agencies, provides a framework to guide the implementation of stormwater best management practices for the development industry and operational best management practices within the City, consistent with the overall objectives and approach of the Plan.

15. Monitoring and Plan Review

A monitoring plan for each study area has been prepared which includes environmental or field monitoring and a “desktop” monitoring component. Environmental monitoring and reporting includes assessing the effectiveness of the works implemented, the degree and rate of improvements in meeting the WWFMMP targets and determining the overall environmental health of the watersheds. Desktop monitoring is intended to track the progress in terms of plan implementation such as the number of roof downspouts disconnected, kilometres of streams restored, number of end-of-pipe facilities commissioned, etc.

The data and analysis derived from this monitoring program will be an integral component of the Plan review process described above.

16. Conclusions

An overview of the development of a 25 year Wet Weather Flow Management Master Plan and the long term (100 year) Preferred Strategy for managing wet weather flows in the City of Toronto has been presented. The overall goal of the Preferred Strategy is to meet Provincial Water Quality Objectives within the City of Toronto surface waters. Through the implementation of the Master Plan, the City is expected to fulfil its legislative requirements and meet corporate objectives of ensuring health & safety, infrastructure protection and accommodating future growth. Further, the Plan recommendations are flexible and can be implemented and adjusted to meet changing corporate priorities and new and emerging technologies.

The measures contained within the Master Plan include: an enhanced public education and community outreach program, enhanced municipal operations, shoreline management, source controls, conveyance controls, end-of-pipe controls, basement flooding protection works, stream restoration works and environmental monitoring and plan review. Overall benefits expected to be derived through the Plan include: swimmable waterfront beaches; control of combined sewer overflows in compliance with legislative requirements; basement flooding protection; protection of the City's infrastructure from stream erosion; restoration of degraded local streams and aquatic habitat and the reduction of algal growth along the waterfront and improved stream water quality in area watercourses.

The cost of implementing the Plan over the 25 years is estimated to be $1.047 billion. The implementation of these new capital works will generate operational and maintenance costs estimated to be $233 million over the 25 years with an estimated $16 million per year by the end of the 25 year plan. The financial impact on the average home owner is projected to be between $30 and $70 per year depending on the funding option implemented.

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NEW REGULATIONS FOR THE TREATMENT OF COMBINED SEWAGE IN AUSTRIA – BASED ON MINIMUM REQUIREMENTS AND WATER QUALITY CRITERIA

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1. Introduction

The Austrian Water Act requires the development of effluent regulations for combined sewer overflows (CSOs) in the competence of the Ministry for Agriculture, Forestry, Environment and Water. Also the EU directive 91/271/EEC concerning urban wastewater treatment contains a requirement for all member states to decide on measures to limit pollution from storm water overflows, but without detailed specifications [1].

The first attempt to regulate CSO discharges was made in the early 1970s, when the International Commission for Lake Constance decided to control the "diffuse" sources of wastewater. Based on the work of Krauth [2] and Swiss experiences a first guideline for CSOs was developed (Bodenseerichtlinie Nr. 14) [3]. The main goal of this work was to make separate and combined sewer systems equal with respect to water protection, at that time mainly based on BOD discharge. This guideline was later transformed to a guideline of the Austrian professional association [4], which contains two basic requirements that can be described in a simplified way as follows:

- Combined sewer overflows without storage capacity must not discharge during rainfall intensities below 15 (30) l/(s·ha), which was called the critical rain intensity, in order to limit the number of overflow events.
- For all CSOs where the flow towards the treatment plant has to be reduced below the minimum flow stated above, this is the case at least at (or just before) the treatment plant, a storage capacity of 15 m^3/ha impervious area has to be provided. The hydraulic capacity of the treatment plants is normally designed for 2 times peak dry weather flow. Today in- and offline storage tanks with bypasses and overflow clarification are the prevailing CSO abatement measure in Austria.

This guideline of the Austrian professional association [4], which requires an extent of CSO abatement measures similar to the German ATV Guideline A-128 [5], will be used until effluent regulations for CSOs will be laid down in an ordinance of the Ministry for Agriculture, Forestry, Environment and Water. The Austrian Water Act claims that specific ordinances have to be issued for all polluted water discharges based on best available techniques as minimum requirements (precautionary principle). The minimum requirements can only be defined as limit concentrations for

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parameters characterising the effluent quality, production specific pollution loads for industrial effluents or percentage pollution reduction requirements but not in the form of construction guidelines (e.g. storage volume, dilution ratio).

For CSO discharges the minimum requirements will be defined as percentage of the annual ammonia load in stormwater runoff (and as percentage of the annual suspended solids load, respectively) that has to be treated biologically in wastewater treatment plants. The effect of the whole CSO concept in a sewer network has to be proved by a combination of hydrological modelling techniques with simplified pollution load calculations. Verification of the model results by full scale monitoring data is also required.

In addition to the minimum requirements, the Austrian Water Act demands that combined sewer systems have to be designed and constructed in a way that the ecological integrity of the river is not significantly disturbed [6]. This corresponds to the good condition of waters required by the E.U. Water Framework Directive. A new version of the guideline of the Austrian professional association will formulate water quality criteria to abate acute water quality problems, such as oxygen depletion or ammonia toxicity.

By now drafts of both regulations the CSO ordinance and the new guideline of the Austrian professional association exist. This paper presents the main aspects of the drafts.

2. Cost benefit considerations

The definition of minimum requirements for CSOs poses the problem that all attempts to find a cost efficiency optimum fail for several reasons. The most important one is that increasing storage capacity for wet weather discharges results in an increasing retention of pollution from the receiving water with a steady decrease in cost efficiency. The second point is that pollution loads from CSOs (containing raw sewage) and from treatment plant effluents and their long term effects on the status of receiving water quality are not comparable in principle and there is no consent on how make them comparable as this is a very complex issue.

The importance of measures to reduce the pollution loads from combined sewer overflows should be assessed on the basis of mass balances considering different system boundaries. The following calculations were done for a hypothetical area with an assumed population density of 100 inhabitants per ha of impervious area. The hypothetical treatment plant was designed according to the German design rules [7]. The hydraulic load of wastewater treatment plants is usually limited to twice the wastewater peak flow, which is 3 to 9 (or more) times the average dry weather flow in most cases. Hence 15–35 % of the yearly stormwater runoff is treated biologically even if there is no storage of stormwater runoff in the sewer system at all. A 10-year rain series with an average yearly rainfall of 774 mm was used for simulation.

Tab. 1 and Tab. 2 show the mass balances for nitrogen and lead. The specific N-load in the wastewater is 4 kg per inhabitant/year and the specific N-load in the surface runoff is 20 kg per ha of impervious area and year. Therefore the relative contribution of surface runoff to the annual nitrogen load in the sewer system is only

5 %, which applies to average conditions in Austria. The nitrogen removal efficiency in the treatment plant was defined as 80%.

Tab. 1 shows the percentage of the annual nitrogen load in the sewer system being discharged to the receiving water by CSOs and by the wastewater treatment plant effluent, depending on the type of sewer system and the size of storage tanks. The nitrogen discharges into the receiving water are almost independent of the type of sewer system and the storage tank volume. Assuming that denitrification in the wastewater treatment plant is affected by stormwater inflow, the difference between the scenarios declines even more. Considering also diffuse sources of nitrogen discharges, the relative contribution of surface runoff to the total discharge into surface waters becomes very small. The total N-loads (including diffuse sources, e.g., from agriculture) to surface waters were estimated as 13 kg per inhabitant and year for Austria in 1995 [8]. Therefore, the relative contribution of surface runoff (or combined sewer overflows) to the total nitrogen loads into surface waters does not exceed 1-2 %.

Table 1. Nitrogen loads to surface waters expressed as percentage of the annual nitrogen load collected in the sewer system.

Nitrogen	Separate system	Combined system			
Storage volume (m^3/ha)	0	0	10	20	40
CSO	5.0	5.2	3.5	2.5	1.5
WWTP effluent	19.0	19.0	19.4	19.5	19.7
CSO +WWTP effluent	24.0	24.2	22.9	22.0	21.2

Table 2. Lead loads to surface waters expressed as percentage of the annual lead load collected in the sewer system.

Lead	Separate system	Combined system			
Storage volume (m^3/ha)	0	0	10	20	40
CSO	50.0	34.2	22.9	17.5	11.1
WWTP effluent	5.0	6.6	7.7	8.3	8.9
CSO +WWTP effluent	55.0	40.8	30.6	25.8	20.0

When the same system is analysed for heavy metal loads (in this case for lead) the benefit of measures reducing stormwater loads becomes much more evident. The relative contribution of surface runoff to the annual lead (or copper, zinc and cadmium) load collected in the sewer system amounts to approximately 50% and the relative contribution of wastewater is also 50% [9]. The removal efficiency for lead in the treatment plant was defined as 90 %. In separate sewer systems heavy metals are primarily accumulated in surface waters, while in a combined sewer system a much higher percentage of the heavy metal load accumulates in the sludge. Storage tanks clearly decrease the lead load discharged to surface waters. But Tab. 2 shows that an increase of the storage tank volume beyond 10-20 m^3/ha influences the Pb loads to surface waters only slightly. It has to be added that the efficiency of the system depends significantly on the removal efficiency in the treatment plant. Considering also diffuse sources (which for lead are in the same order of magnitude as point sources) the relative contribution of CSOs to the total lead loads discharged to surface waters becomes smaller, but still remains significant.

Combined sewer overflows discharge raw wastewater to the receiving waters. Without any storage in the sewer system approximately 2 - 4% of the annual wastewater quantity does not reach the treatment plant. With a storage volume of 10 m^3/ha impervious area this amount is reduced by 35%, and for particulate pollutants by 50%. Again doubling the storage volume reduces the raw wastewater discharges by no more than further 15%.

These mass balances show that CSO treatment significantly decreases the discharges of raw wastewater, suspended solids and heavy metals from urban sewer systems to surface waters. For this reason it is not advisable to design CSOs on the basis of water quality criteria alone. However, the minimum requirements for CSO design have to be characterised by a reasonable cost-benefit ratio.

On one hand the efficiency of stormwater tanks decreases with increasing volume (per ha impervious area). On the other hand the specific costs per m^3 of stormwater tank volume decrease with increasing volume. Fig. 1 shows the cost-benefit ratio for storage tanks in regard to the annual discharge volume (corresponding to the dissolved pollution load) and the discharged load of suspended solids. The cost-benefit ratio increases with increasing storage tank volume and reveals no optimum and no point where an increase in volume becomes significantly less efficient. It is widely accepted in Germany and in Austria that specific storage tank volumes exceeding 40 m^3/ha are not efficient. But on the basis of cost-benefit curves it is possible to select 10 m^3/ha as well as 30 m^3/ha.

Based on these cost benefit considerations it was recommended to initially set low requirements (e.g. specific volumes of storage tanks not exceeding 10-15 m^3/ha impervious area) for CSOs located at less sensitive (“normal”) waters, but to make provision for potential upgrading. However, this decision was also motivated by the wish of the public authorities to continue with the level of stormwater treatment used so far.

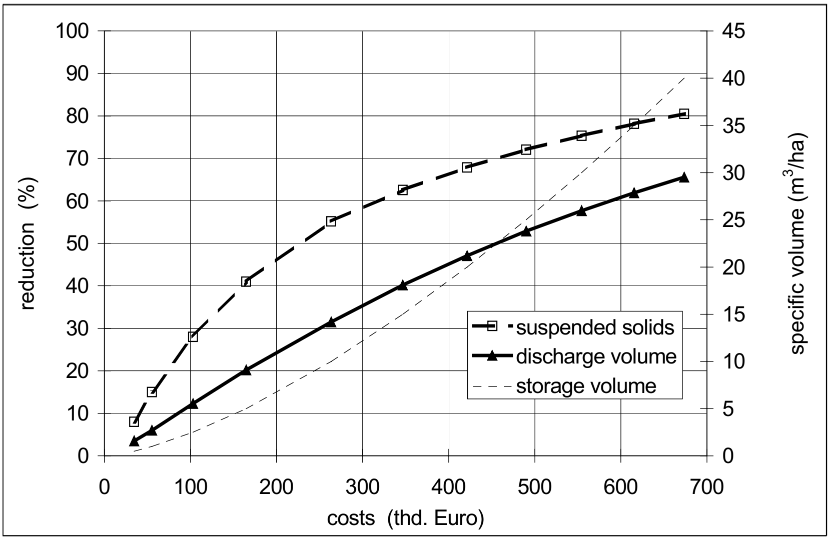

Figure 1. Cost-benefit-curves for stormwater storage tanks

3. Minimum requirements

The draft regulation on combined sewer overflows defines minimum percentages of annual loads contained in the wet weather flow which have to be treated biologically in the wastewater treatment plant.

Table 3. Percentage of ammonia and suspended solids loads in urban runoff, which has to be treated biologically in the wastewater treatment plant [10].

		Size of the wastewater treatment plant (EW)		
		<5.000	5.000 - 50.000	> 50.000
1	NH_4N	55	60	65
2	AFS	70	75	80

The minimum requirements are defined as:

$$\eta = \frac{(VQ_r \cdot c_r - VQ_{cso} \cdot c_{cso})}{VQ_r \cdot c_r} \cdot 100$$

η efficiency for ammonia and suspended solid loads in stormwater runoff (%)
VQ_r annual stormwater runoff volume (m^3/y)
VQ_{cso} annual volume of combined sewage discharged to receiving waters (m^3/y)
c_r concentration in stormwater runoff in the combined sewer system (mg/l)
c_{cso} concentration in combined sewer overflow (mg/l)

The requirements specified in Tab. 1 apply to regions with an average yearly precipitation (h_N) smaller than 600 mm. For combined sewer systems in an area with higher annual precipitation the following reductions of the required efficiency can be set:

for h_N more than 600 mm but not more than 800 mm, 5%
for h_N more than 800 mm but not more than 1200 mm, 10%
for h_N more than 1200 mm, 15 %.

As ammonia is a dissolved compound the regulations for ammonia guarantee that a minimum percentage of stormwater runoff in the sewer system has to be transmitted to the wastewater treatment plant. The required efficiency for suspended solids was set 15% higher in order to force mechanical treatment of the discharged combined sewage.

When fixing the minimum percentages the following factors were taken into account:

a) Annual precipitation
b) Inhabitant density related to the impermeable surface
c) Separate sewer systems connected to the combined sewer system

3.1 ANNUAL PRECIPITATION

The higher the annual precipitation is, the larger the storage volume has to be, in order to achieve a uniformly demanded efficiency. In Austria the average annual precipitation varies within a very wide range from <500 mm to >2000 mm. The definition of a uniform minimum efficiency independent of the annual precipitation depth would therefore lead to very different tank volumes. However, it appears not meaningful to establish very large combined sewer overflow tanks in areas with high annual precipitation since in this case a high amount of "clean" rainwater is stored. Therefore an adjustment of the requested efficiencies was made according to the annual precipitation, with the result that the extent of the necessary measures is independent of the local annual precipitation to a large extent.

3.2 INHABITANT DENSITY RELATED TO THE IMPERMEABLE SURFACE

The smaller the inhabitant density related to the impermeable surface is, the larger the specific storage volume (m^3/ha impervious area) has to be, in order to achieve the required efficiency. The inhabitant density is usually smaller in rural municipalities than in cities. Therefore combined sewer overflow tanks with a substantially larger specific volume would have to be established in rural municipalities than in cities, if in both cases the same efficiency of stormwater treatment were to be achieved. However, it appears not meaningful to establish tanks with a substantially larger specific volume in rural areas because the benefit for water protection achieved thereby is small in relation to the costs. In addition the concentrations are usually lower in the discharged combined sewage if the inhabitant density is small, since the ratio between rain discharge and dry weather discharge is in that case higher. Therefore the regulation contains a classification in 3 requirement classes in dependence on the size of the associated wastewater treatment plant. Thus the extent of the required measures is independent of the inhabitant density to a large extent.

3.3 SEPARATE SYSTEMS CONNECTED TO THE COMBINED SYSTEM

If a separate sewer system is connected to the combined system, the concentrations in the discharged combined sewage tend to be higher than in the original combined system. In order to maintain the discharged loads on the level of the original combined system a higher percentage of wet weather flow has to be treated in the wastewater treatment plant. As a function of the relation between the average dry weather discharge from the separate system to the average dry weather discharge from the combined system the required efficiency has to be increased by

$$5 \cdot \frac{Q\,(dry\ weather\ flow - seperate\ \ sewer\ \ system)}{Q\,(dry\ weather\ flow - combined\ \ sewer\ \ system)} \cdot 100 \quad (\%)$$

but not more than up to 75% for NH4-N and up to 90% for SS.

In contrast to the existing guidelines in Austria, which define a critical flow rate of 7–15 l/(s·ha impervious area) for CSOs without storage and a minimum stormwater tank volume of 15 m³/ha impervious area, the new regulation gives the engineer the possibility to achieve the required level of combined sewage treatment by means of a number of different measures. In addition to storage this includes e.g., the disconnection of impervious surfaces from the sewer system, the increase of the WWTP's inflow and (pollution based) real-time control strategies for combined sewer systems. The minimum efficiencies set by the draft regulation have to be applied for the entire catchment of the combined sewer system but not for single combined sewer overflow structures. A minimum dilution rate of 1:8 is the only requirement for each CSO. This will make planning more flexible compared to the present situation.

3.4 CALCULATION OF MEASURES IN ORDER TO ACHIEVE THE EMISSON BASED MINIMUM REQUIREMENTS

3.4.1 Minimum efficiencies for NH_4N

The percentage of the stormwater runoff in combined systems which is treated in the wastewater treatment plant within one year (i.e., an average year) has to be calculated with the help of a long-term simulation.

$$\eta_r = \frac{(VQ_r - VQ_{cso})}{VQ_r} \cdot 100$$

η_r efficiency for stormwater runoff in the combined sewer system (%)
VQ_r annual volume of stormwater runoff in the combined system (m³/y)
VQ_{cso} annual volume of combined sewage discharged to receiving waters (m³/y).

The efficiency η_r corresponds to the required minimum efficiency for NH_4-N since ammonia is a dissolved compound.

Calculation of the percentage of urban runoff discharged to receiving waters by CSOs has to be done with a long term simulation of the runoff processes in the sewer system using a historical rainfall record covering at least ten years. In order to guarantee a minimum level of model verification at least duration and frequency of overflow events have to be measured at the most relevant CSOs.

3.4.2 Minimum efficiencies for suspended solids

The simulation shows, which part of the stormwater runoff in the combined sewer system is discharged to receiving waters at every single combined sewer overflow and at every single combined sewer overflow tank.

With combined sewer overflows the "water quantity efficiency" corresponds to the efficiency for suspended solids, since neither storage (of flushes) nor a sedimentation effect is given. However, with combined sewer overflow tanks the efficiency regarding suspended solids goes beyond the efficiency for ammonia. According to the draft of the new guideline of the Austrian professional association the following settling efficiencies can be assumed for sedimentation tanks. The same values can be applied to detention tanks, when first flush phenomena are likely to occur.

$$\eta_{sed} = \frac{(c_r - c_{cso})}{c_r} \cdot 100$$

η_{sed} settling efficiency (%)
C_r concentration in stormwater runoff of the combined sewer system (mg/l)
C_{cso} concentration in combined sewer overflow (mg/l).

Table 4. Sedimentation efficiency η_{sed} for suspended solids as a function of the storage volume

specific Volume (m^3/ha $_{impervious\ area}$) Sedimentation tank	“storage sewer”	Efficiency η_{sed} (%) for suspended solids
0	0	0
5	10	20
10	20	35
15	30	50

The final efficiency for suspended solids can be calculated in a simplified form as follows:

$$\eta_{ss} = \eta_r + \frac{VQ_{cso,tank}}{VQ_r} \cdot \eta_{sed}$$

η_{ss} efficiency for suspended solids (%)
η_r efficiency for stormwater runoff (%)
η_{sed} settling efficiency (%)
$VQ_{cso,tank}$ annual volume of combined sewage discharged to receiving waters at storm water tanks (m^3/y)
VQ_r annual volume of stormwater runoff (m^3/y).

It is also possible to calculate the efficiency for suspended solids by pollution based long-term simulation.

4. Ambient water quality

Combined sewer overflows may cause several short term adverse effects on receiving waters, including dissolved oxygen depletion, ammonia toxicity and reduced biodiversity. A new version of the guideline of the Austrian professional association will define criteria in order to identify critical cases, in which the minimum requirements might not be sufficient to prevent the aquatic biota from being impaired significantly. This paper shows the approach for hydraulic stress, ammonia and oxygen depletion.

4.1 HYDRAULIC EFFECTS

Combined sewer overflows change the flow conditions especially in small streams within very short time. The high hydraulic load can lead to drifting of organisms from its habitats and thus to the local loss of individuals. In extreme cases CSOs can result in wide movements of the bed sediments with a comprehensive transport of organisms [11]. The effect of the hydraulic load depends substantially on the river morphology. How quickly an impaired section recovers again, depends on the presence of unaffected lateral waters, since then disturbances are faster mitigated by colonisation. Investigations in Switzerland demonstrated that an ecological impairment is to be expected, if the total number of events with bed movement is doubled (or more) by combined sewer overflows [12].

The draft of the new guideline requires to examine in detail the necessity of additional retention measures, if the maximum discharge from the sewer system ($Q_{e,1}$) (determined by calculation for a rain event with a return period of one year) exceeds 10 to 50% of the mean high discharge of the river (HQ_1). The lower value of 10% applies to waters with predominantly sandy sediment, small width variability and low river rehabilitation potential. The higher value of 50% applies to waters with stony sediment, high width variability and high river rehabilitation potential. For the calculation the sum of all discharges within a defined river section must be considered.

Due to investigations [11, 13, 14] it can be assumed that the hydraulic load of rivers caused by combined sewer overflows does not cause substantial effects on the biocoenosis, if $Q_{e,1}$ is lower than $0.5 \cdot HQ_1$.

If the relation between the maximum discharge from the sewer system ($Q_{e,1}$) and the mean high discharge of the river (HQ_1) is larger than indicated above, the necessity for retention of the combined sewer discharges has to be examined in more detail. The probability of significant effects of the hydraulic load on the biocoenosis should be judged by benthic community assessments (taxa richness and total counts of benthic organism upstream and downstream of the overflow).

However, the decision, which degree of deviation is a "significant effect" and therefore produces the need for further measures, is the result of a political consensus considering ecological, technical and economic aspects. Not every effect on an ecosystem is a significant impact. Investigations at several small rivers in Austria clearly showed that in the majority of cases it is difficult to identify impacts caused by the increased flow from CSOs as other factors (primarily the river morphology) prevail.

4.2 AMMONIA TOXICITY

Un-ionized ammonia is a nerve poison for higher organisms. For salmonid fish, which belong to the most sensitive organisms, lethal concentrations ($LC_{50,24\text{-}96h}$) vary between 0.08 mg NH_3/l and 3 mg NH_3/l [15,16]. The toxicity partially depends on factors such as the age and condition of the fish, whereby the spawn is generally most sensitive. The pH value seems to have a substantial influence on the toxicity of ammonia, whereby the negative effect of the NH_3 concentration decreases with rising pH value. Some authors point to a significant influence of the temperature on the ammonia toxicity [17]. According to their investigations the sensitivity of fish significantly decreases with rising temperature.

The draft of the guideline of the Austrian professional association requires that total ammonia concentrations in rivers must not exceed 2.5 mg/l NH_4-N more frequently than once a year in order to protect sustainable salmonid fishery and 5 mg/l NH_4-N for sustainable cyprinid fishery. At a pH value of 8 and a temperature of 20°C the corresponding concentrations of un-ionized ammonia are 0.1 mg/l NH3-N and 0.2 mg/l NH_3-N respectively. Higher pH values and temperatures are unlikely to occur in rivers after mixing with the discharged combined sewage.

Critical cases can be identified by calculating the NH_4-N concentration in the river at different mixture ratios. The total ammonia concentration of stormwater runoff is usually not higher than 1 mg/l NH_4-N. The probability of toxic events can be estimated with higher accuracy by simulations. On behalf of the Swiss professional association VSA [18] a “simple” model was developed, which calculates the resulting ammonia dose (concentration x exposure time) for each individual overflow event and compares this value to the dose critical for fish.

Given a total ammonia concentration the resulting concentration of un-ionized ammonia depends considerably on the pH value. An increase in the pH value of approximately 0.3 units leads to doubling the un-ionized ammonia concentration. Measurements of pH in a small creek (Petersbach) near Vienna showed that the discharge of combined sewage usually leads to a reduction of the pH value in rivers directly downstream of the overflow, since the pH value of the combined sewage is often lower than that of rivers. The pH value, which results from chemical equilibrium due to the mixture of discharged combined sewerage and river water, can be calculated. However this "mixture pH" does not remain constant in further flow processes. On one hand CO_2 exchange with the atmosphere causes a pH-value rise on the other hand CO_2-production due to the discharged organic load causes a pH-value fall. Thus the measuring point should not be located directly downstream of the discharge point, since the pH value can change rapidly again, while the total ammonia concentration in rivers changes slowly due to nitrification and hydrolysis.

Photosynthesis has a substantial influence on the pH value in rivers. Fig. 2 shows that combined sewer overflows dampen the diurnal fluctuation of the pH due to smaller light intensity (cloudy weather) and the turbidity of rivers during rain events.

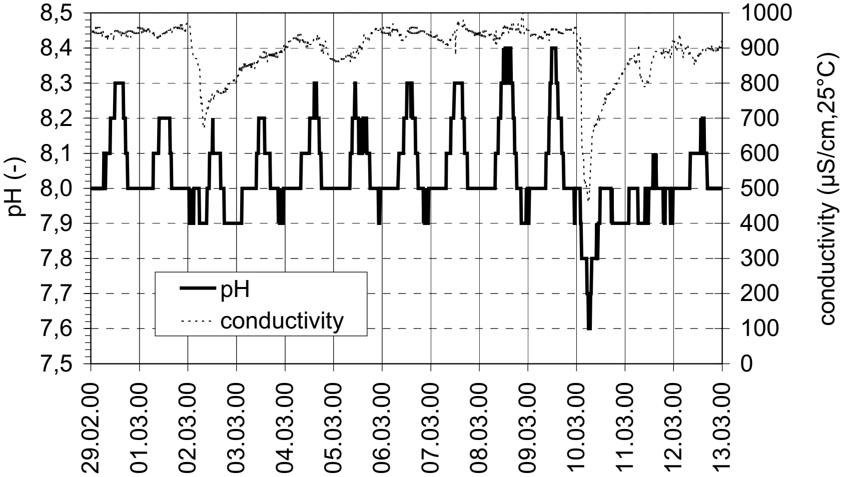

Figure 2. pH and conductivity measured in a small creek near Vienna, 4 km downstream from the CSO [19].

4.3 DISSOLVED OXYGEN DEPLETION

According to the draft guideline of the Austrian professional association the short time (hours) mean oxygen concentration (DO) in rivers must not be lower than 5 mg/l more frequently than once a year. Oxygen depletion caused by CSOs is not a prevalent problem in Austria. In the majority of cases only very small creeks with low flow velocities are endangered.

In shallow rivers, oxygen depletion results mainly from degradation by sessile organisms. The amount of sessile organisms depends very much on the existence and quantity of continuous organic loads under dry weather conditions. This could be observed in the small creek Petersbach near Vienna as well. The measurements of DO revealed that the creek was organically loaded under dry weather conditions. From May 15th to May 17th DO varied between 7 and 9 mg/l due to photosynthesis. During the day the creek was oxygen-saturated (9 mg/l at 20°C), at night an oxygen deficit of approximately 2 mg/l developed. A thunderstorm in the evening of May 18th (in Fig. 3 identifiable by the decrease of the pH value to 7.8) led to a short term (<1 h) decrease of DO by approximately 2 mg/l to 5.3 mg/l. The combined sewage discharge probably resuspended deposits (pollutants and biomass) already existing before the rain event, which then caused the oxygen deficit. Similar measurements in other rivers (creeks) indicate that the effect just described is likely to occur in shallow, slow flowing creeks.

Particularly in shallow rivers the description of all processes influencing the oxygen consumption (i.e. degradation, of soluble and particulate organic matter in the water phase, sedimentation and extraction to the sediments, degradation of organic matter by sessile biomass, resuspension of settled material) and the assessment of all parameters involved is rather complex. Thus it is difficult to identify critical cases with simulation of the dissolved oxygen concentration alone. The guideline therefore recommends on-line measurements of DO for rivers with a gradient lower than 3-5 m/km when phenomena like anaerobic sediments and/or oxygen deficit under dry weather conditions are observed.

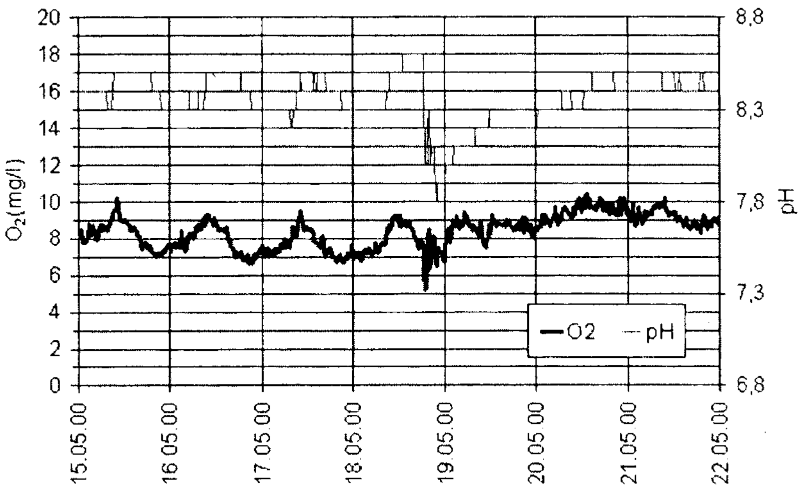

Figure 3. DO and pH measured in a small creek near Vienna, 4 km downstream from the CSO outfall [19].

5. Conclusions

The minimum requirements for receiving water protection from combined sewer overflows laid down in the draft of the new regulations for Austria result in a significant reduction of the discharges of raw wastewater, suspended solids and heavy metals from urban sewer systems. For this reason it is not advisable to design CSOs on the basis of water quality criteria alone. However the minimum requirements for CSO design have to be characterised by a reasonable cost-benefit ratio. It was recommended to initially set low requirements (e.g. minimum efficiencies requiring specific volumes of storage tanks not exceeding 10-15 m^3/ha impervious area) for CSOs located in less sensitive ("normal") waters, and to make provision for potential upgrading. Development of minimum requirements in form of percentage load reduction requirements (instead of construction guidelines) for the entire catchment (instead of single CSO structures) makes planning significantly more flexible.

A new version of the guideline of the Austrian professional association will define criteria (which can be calculated or measured with reasonable effort) in order to identify critical cases, in which the minimum requirements might not be sufficient to prevent the aquatic biota from being impaired significantly. However, combined sewer overflows are often located in urbanised rivers and investigations in several small rivers in Austria clearly showed that under these conditions aquatic organisms get impacted by other factors than CSOs. Thus in many cases it might be much more effective to improve the river morphology and to reduce permanent dry weather discharges than to invest in large stormwater tanks in order to abate acute water quality problems.

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IMPLEMENTATION EXPERIENCE OF CSO POLLUTION CONTROL POLICIES AND PROCEDURES IN THE UK

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1. Introduction

Historically, combined sewer overflows (CSOs) in the UK have been designed to discharge at fixed multiples of the dry weather flow in the sewer, usually 6 x DWF. Flows in excess of 6 x DWF would be spilled and the remainder carried forward to a Wastewater Treatment Plant (WTP). Problems arising from this pragmatic approach were recognised as early as the 1960s when a new approach, 'Formula A' [1], was proposed to calculate an acceptable pass forward flow from a CSO. This is still in use as a minimum CSO spill setting criteria. In the 1960s, new designs of overflow were adopted to provide for retention of pollution in the sewer, such as the use of storage, and stilling pond or high side weir overflows. A method for estimating storage requirements was developed in the late 1970s [2]. Referred to as the SDD method, it is based on a Formula A spill threshold, plus storage within the CSO related to the available dilution in the river and the upstream sewer population equivalent. Neither the Formula A nor the SDD approach can be directly related to compliance with receiving water standards.

The last decade has seen a progressive development in technical know-how and cost-effective planning and evaluation tools for use by the UK water industry. The problems are now being addressed through the planning framework provided by the Urban Pollution Management (UPM) Manual procedure [3]. This procedure has enabled the environmental regulators and dischargers to address a backlog of improvements to WTPs and CSOs.

A previous paper [4] considered how the UPM procedure is being used to assist in the delivery of a major CSO improvement programme in England and Wales, where the water industry has been privatised. This paper considers two UPM studies in Northern Ireland. The locations of the study catchments, West Belfast and Cookstown, are shown in Figure 1. The studies employed the UPM procedure, as described in the Second Edition of the UPM Manual [5], through the use of simplified, integrated, urban wastewater modelling tools. The basic tools were modified and developed as appropriate to the needs of each study to assess the impacts of CSO and other wet weather discharges in both inland and tidal receiving waters. Both studies reflected the 'partnership' approach promoted by the UPM Manual [5]. The studies were jointly undertaken by WRc and Black and Veatch

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working as respectively consultants to Environment and Heritage Service (EHS) and Water Service (WS), both part of the Government in Northern Ireland.

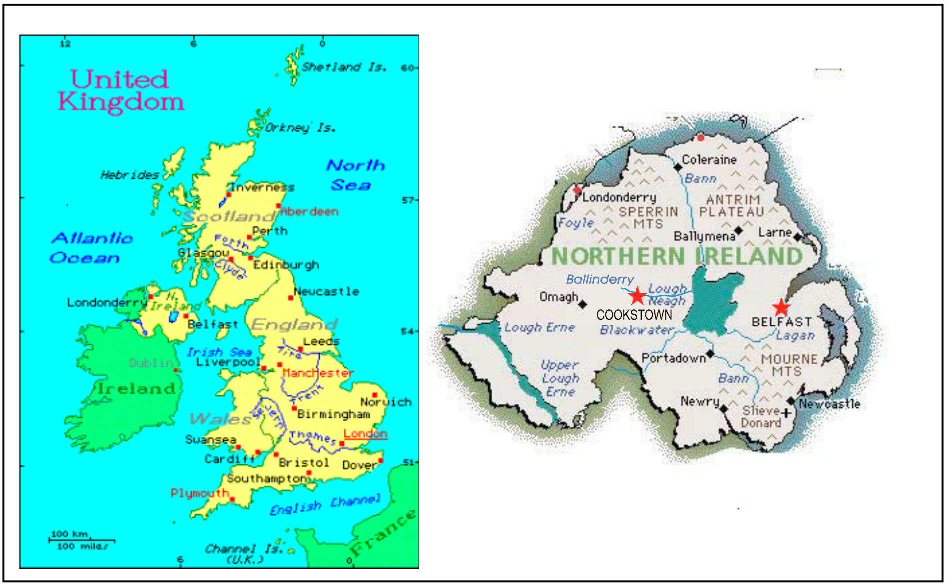

FIGURE 1. Location of West Belfast and Cookstown in Northern Ireland

2. Organisation and Environmental Regulation of the Water Industry in Northern Ireland

In Northern Ireland, water supply, wastewater transport and wastewater treatment are provided by WS. Environmental regulation is the responsibility of EHS. A core activity of EHS is to protect, maintain and, where appropriate, to improve the quality of rivers in Northern Ireland. In planning river quality management and pollution control strategies, EHS issues registered standards for WS's WTP discharges. There are over 900 WTPs for a total population of 1.6 million people in Northern Ireland. Over 600 WTPs serve a population of less than 250. In addition, there are estimated to be around 2800 CSOs, including emergency overflows at sewage pumping stations. EHS has been involved in the development of the UPM procedure and requires its use to support a programme for CSO improvements in Northern Ireland. The two case studies illustrate the minimum UPM study approach required by EHS to suit environmental and economic conditions in Northern Ireland.

3. West Belfast UPM Study

The city of Belfast is the largest population centre in Northern Ireland and is situated at the head of Belfast Lough. The study area comprises the West Belfast sewer catchment draining to the West Belfast WTP. The study area, shown in Figure 2, covers some 40 km^2 in the centre of the city and comprises a mix of residential, commercial, industrial and amenity areas. The main watercourses within the study area are the tidal River Lagan and its major tributary the Blackstaff River, together with an extensive network of smaller watercourses draining from the hills to the west of the city centre to the River Lagan and Blackstaff River. The older, central area of West Belfast is served by two large diameter brick trunk sewers constructed during the period 1888 to 1896. These sewers run parallel to the west of the River Lagan. As the city developed during the 20th Century, the original trunk sewers became overloaded and a third large diameter concrete trunk sewer was constructed during the period 1960 to 1972. The continued development and expansion of Belfast during the 20th Century has created a situation where the sewer system is no longer hydraulically capable of adequately retaining and conveying storm flows. A total of 106 CSOs have been constructed to relieve the hydraulic overloading on the system during storms and restrict the incidence of out-of-sewer flooding.

FIGURE 2. The West Belfast Sewer Catchment

In the past, the polluting effects of the CSO discharges were partly mitigated by the flushing effect of tidal flows in the River Lagan. However, the construction of the new Lagan Weir in 1993 extended the length of the Lagan impoundment to include the discharge from the Blackstaff River, thereby increasing the total polluting load from CSO discharges to the impoundment.

Improvement to the water quality and visual amenity of the impounded tidal reach of the River Lagan flowing through the centre of Belfast is a major component of the city's regeneration plans. The impoundment fails to achieve its designated target of a depth averaged concentration of 4.0 mg/l dissolved oxygen, expressed as a long-term 5%ile. The impact of wet weather CSO discharges are thought to be the main cause of failure to meet the dissolved oxygen standard. Modelling has shown that the CSOs spill an average of some 3.7 million cubic metres of storm sewage either directly to the River Lagan, or indirectly via the Blackstaff River and the minor tributaries. Consequently, the operation of the CSOs has become a major cause for concern.

An engineering study of the Belfast sewer system, completed in 1994, recommended an upgrading option with an estimated cost between £100 and £120 millions. The principal features of the proposed scheme comprised:

- 10.4 km long tunnelled interceptor sewer of 1.2 m to 4.0 m diameter with 16 m^3/s capacity terminal pumping station;
- closure of 27 CSOs;
- improvement of the remaining 79 CSOs to pass forward increased flows to treatment and provide screening to retain aesthetically polluting solids;
- 9,500 m^3 of local storage to limit CSO spill frequencies and volumes; and,
- local structural and hydraulic upgrading works.

A major requirement of a review of the original study, commenced in 1998, was a more detailed assessment of the impact of the CSO discharges on the Blackstaff River and the River Lagan. The UPM procedure [5] was employed to undertake the analysis of the CSO impact. This was carried out for the Blackstaff River using the SIMPOLv2 integrated modelling tool [5]. The results from this part of the study were then used to provide inputs to a further study to assess the impact of the combined CSO and Blackstaff River discharges on the tidal River Lagan impoundment, using SIMPOL3 [6]. This is more advanced version of the SIMPOLv2 integrated modelling tool. A SIMPOL3 model is a dynamic, deterministic hydrological and water quality model that is driven by time-series input data and produces time-series results that can then be compared with water quality standards. A full description of the West Belfast UPM study is available elsewhere [7].

While the Belfast sewer system is relatively large and complex, it was possible to conceptualise the system as 17 hydraulically independent subcatchments using SIMPOLv2. Models were developed to represent both the existing and future, upgraded sewer systems. The SIMPOL2 sewer module was hydraulically calibrated using a detailed HydroWorks model that had been developed during the earlier study. Quality calibration was achieved using observed data collected at selected CSOs throughout the sewer system during the original fieldwork programme in 1993. The Blackstaff River was represented in the SIMPOLv2 river module as four reaches. Historical data were available to provide estimates of base flows and quality in the tributary streams for the present situation.

Following the UPM procedure, a 10-year time series of rainfall events was synthetically generated for the Belfast study using the STORMPAC stochastic rainfall generator software package [5]. This provided a total of 1768 individual rainfall events with hourly profiles for the subsequent analyses using SIMPOLv2. Four modelling scenarios were simulated:

Scenario 1: The existing situation, with neither the sewer system nor the quality of the flows in the upstream tributaries improved.

Scenario 2: The sewer system improved by implementation of the proposed upgrading scheme, but the upstream tributary quality unchanged.

Scenario 3: The sewer system unimproved, but the quality of the incoming tributary flows improved to the mid-point of the target quality class (by undefined measures beyond the scope of the sewerage upgrading proposals).

Scenario 4: The final situation, where both the sewerage upgrading scheme has been implemented and the quality of the incoming tributary flows has been improved to mid-point of the target quality class (by undefined measures beyond the scope of the sewerage upgrading proposals).

The results from each modelling scenario were evaluated against a variety of environmental standards used in Northern Ireland for the management of river water quality. The main criteria against which the quality of the River Blackstaff was assessed was the FIS for a Sustainable Cyprinid Fishery Ecosystem [5]. Compliance with EHS's river chemical class criteria and equivalent 99%iles was also assessed. The compliance assessment results are summarised in Table 1**.** The number in the results cells represents the average number of 6-hour time blocks per year for which the river reach was non-compliant with the particular standard threshold in each of the four scenarios. The standards against which compliance has been assessed are given at the head of each column, with the numerical threshold value appropriate to the Blackstaff River's target class given underneath. The final header row indicates the number of 6-hour time blocks that may exceed the threshold before failure of the standard is deemed to have occurred.

TABLE 1. River Blackstaff SIMPOLv2 Results

		90%ile		**99%ile**		**10%ile**	**DO**		**un NH3**	
		BOD	**NH4**	**BOD**	**NH4**	**DO**	**1 yr**	**1 mth**	**1 yr**	**1 mth**
	Threshold (mg/l)	8	3	19	6	50% sat	4.0	5.0	0.150	0.075
	Permitted failures	146	146	15	15	146	1	12	1	12
R 1	Scenario 1	681	77	219	6	146	2.3	46.8	2.6	22.4
	Scenario 2	661	77	191	6	146	2.3	47.2	2.6	22.7
	Scenario 3	179	10	26	0	150	0.0	4.8	0.0	1.6
	Scenario 4	132	9	5	0	150	0.0	4.7	0.0	1.6
R 2	Scenario 1	638	76	185	6	143	1.9	44.0	1.4	17.3
	Scenario 2	605	76	161	6	142	2.0	43.7	1.4	17.2
	Scenario 3	172	9	18	0	143	0.0	5.2	0.2	1.9
	Scenario 4	108	9	4	0	145	0.0	5.2	0.2	1.9
R 3	Scenario 1	645	38	263	0	170	4.4	50.8	0.5	7.4
	Scenario 2	572	33	174	0	159	3.4	34.9	0.4	6.5
	Scenario 3	203	18	85	1	151	0.50	18.9	0.5	4.4
	Scenario 4	74	16	5	1	145	0.3	10.6	0.6	4.2
R 4	Scenario 1	561	500	179	66	746	332.6	719.5	98.3	233.9
	Scenario 2	479	475	83	65	668	264.3	627.3	77.9	195.0
	Scenario 3	254	70	93	2	345	63.9	209.5	10.5	33.1
	Scenario 4	132	63	5	2	233	12.1	99.0	0.6	8.9

Fails standard , R = reach

The conclusions from examination of the results from the SIMPOLv2 modelling were:

- The Blackstaff River currently fails to meet its long-term water quality objective by a substantial margin (Scenario 1).

- The volumes of CSO spills and the associated polluting loads discharging to the impounded River Lagan will be reduced by 68% and 87% respectively following implementation of the proposed sewerage upgrading scheme.
- The Blackstaff River is predicted to achieve its long term water quality objective if the proposed sewerage upgrading scheme is implemented and the upstream tributaries are improved to the mid-point of their quality target classes (Scenario 4).
- There is little potential for reducing the scope of the proposed sewerage upgrading scheme.
- Additional engineering works to control the CSO spills would have very little additional benefit.

Following the finalisation of the sewer upgrading proposals, the next phase was to quantify the improvements that would result in the River Lagan and to assess if the proposed scheme could allow the target dissolved oxygen criterion to be achieved. A detailed 3-dimensional model of the tidal River Lagan was developed in the early 1990's as part of the original studies to design the Lagan weir [8]. As with the impact assessment of the Blackstaff River, it was impractical to carry out the long term statistical assessment against the 5 percentile dissolved oxygen standard using such a detailed model. Therefore, the original model was used to support a SIMPOL3 approach to carry out the impact assessment. The study methodology was again based on the UPM procedure [5], in this case suitably modified to account for the tidal conditions within the River Lagan impoundment. The major steps in the SIMPOL3 applications methodology were as follows:

Step 1: River submodels were built to represent the Blackstaff River and the upstream reaches of the Lagan. These submodels were calibrated against long term historical time series data for river flow and quality.

Step 2: Sewer submodels were developed, based on those used for the Blackstaff River study, and extended to include those CSO subcatchments discharging directly to the River Lagan both before and following upgrading of the sewer system. The models were recalibrated against detailed outputs from the HydroWorks model of the sewer system.

Step 3: A simplified, depth-averaged, 1-dimensional estuary submodel was developed to represent the hydrodynamic and dissolved oxygen processes in the semi-tidal estuary. The original, detailed, 3-dimensional model was run to simulate dissolved oxygen conditions in the river for 10 selected event conditions for a variety of states of the tide and upstream flow and quality. The results from these detailed model runs were used to calibrate the simplified model.

Step 4: The sewer, upstream river and estuary submodels were combined within SIMPOL3 into two integrated models – one for the current system (Scenario 1), and one for the upgraded future system (Scenario 4). These integrated models were run with complete years of data to represent dry, average and wet rainfall years, selected from the 10 year STORMPAC rainfall time series for Belfast. Variations in tidal conditions, temperatures and upstream water quality were included. The results from these runs enabled the 5 percentile dissolved oxygen values to be calculated at key locations. These values were compared to the target estuary standards, initially for a baseline set of conditions and then for a series of sensitivity runs.

A typical set of summer dissolved oxygen time series results at a single site, the Ormeau Embankment (Lagan unit 6) below the Blackstaff River confluence, is presented in Figure 3. These results are for both the current and future systems under the baseline conditions for the average rainfall year generated by STORMPAC (year 2065). Baseline conditions assumed that there was no photosynthesis/respiration; that there was a minimum sediment oxygen demand; that the quality of the tributary inputs to the Blackstaff River had been improved; and, that the aerators presently installed in the Lagan impoundment were in operation. The results show a general improvement in dissolved oxygen levels for the future system. In particular, there is a reduced drop in dissolved oxygen levels experienced during storm events due to the improved wet weather quality in the Blackstaff River. The detailed results were processed to give key statistics to quantify the overall improvement achieved. A typical example of these results is presented in Figure 4.

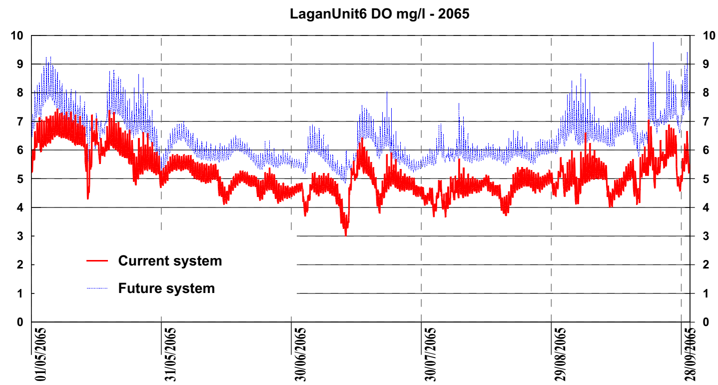

FIGURE 3. An example of SIMPOL3 predicted dissolved oxygen concentrations in the Lagan impoundment during average summer rainfall

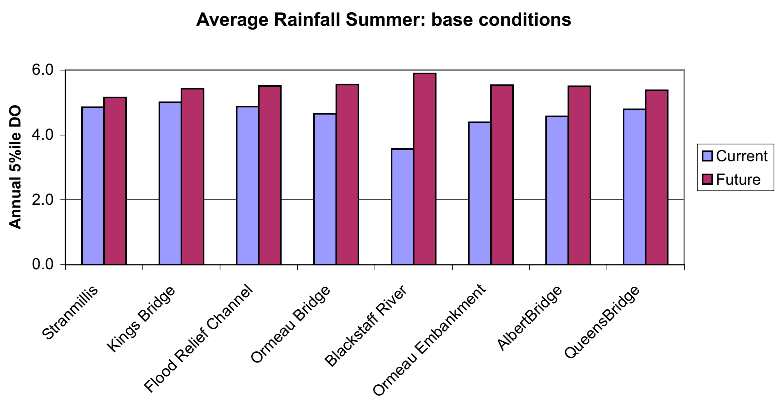

FIGURE 4. SIMPOL3 predicted dissolved oxygen concentration statistics in the River Lagan impoundment for average summer rainfall

The results show a general improvement in dissolved oxygen levels throughout the impoundment, with the greatest improvement being achieved in the lower reach of the Blackstaff River. Within the main body of the impoundment the greatest improvement is in the Ormeau Embankment area that receives the input from the Blackstaff River. Sensitivity runs were carried out by applying a degree of photosynthesis/ respiration; using poorer upstream quality for the tributaries of the Blackstaff River; turning off the aerators; and, doubling the sediment oxygen demand. None of these changes caused the impoundment to fail its quality objective. The proposed sewer upgrading will cause a significant improvement in dissolved oxygen levels throughout the Lagan impoundment that should allow it to achieve its river quality objective.

The overall conclusion from the West Belfast UPM study is that the sewer upgrading proposals will allow the major watercourses running through the city to achieve their long-term water quality objectives, provided additional works are undertaken further upstream to ensure that the streams entering the city are of an acceptable quality.

4. The Cookstown UPM Study

Cookstown is a small town of approximately 15,000 population located on the north bank of the Ballinderry River some 10 km. due west of Lough Neagh. The area is subject to substantial increase in the resident population, which is anticipated to grow to approximately 17,000 during the planning horizon up to 2030. The town is currently served by a combined sewer system, that incorporates 15 CSOs. Sewage flows drain to the Killymoon WTP sited on the north bank of the river. The majority of the CSOs discharge into minor watercourses situated to the west or east of the town. Both of these minor watercourses, known as the Eastern and Western Ditches, flow into the Ballinderry River. Only the WTP effluent, the storm tank overflow and one CSO immediately upstream of the STW discharge directly to the Ballinderry River.

The Ballinderry River runs just to the south of the Cookstown urban area and, following its confluence with several significant tributaries, eventually flows into Lough Neagh. The river is an important habitat for indigenous salmonid species. The Ballinderry River has a river quality objective of river chemical Class B "Good Quality" throughout the study area. This standard is currently achieved upstream of Cookstown, but downstream of Cookstown, the river quality falls to Class D "Fair Quality". Two minor water courses (Eastern and Western Ditches) have extremely low summer base flows and do not have specific river quality objectives.

The study, described in detail elsewhere [9], was undertaken using data that were, for the most part, already available at the start of the study and involved the construction of one new, simplified (SIMPOLv2) model. The purpose of the UPM study was to ensure that proposed upgrading works to both the sewer system and the treatment works would support achievement of a high quality sustainable salmonid fishery in the Ballinderry River below Cookstown. EHS required that, in the future, in addition to meeting the GQA targets, the Ballinderry should be capable of meeting the UPM FIS for dissolved oxygen and unionised ammonia for a Sustainable Salmonid Fishery [5].

Prior to commencement of the UPM study, WS had identified a scheme of proposed improvements to both the sewer system and the sewage treatment works. In summary, these proposals comprise:

- construction of several in-line storm sewage detention tanks within the sewer system;
- separation of substantial areas of combined sewerage and the construction of new storm sewers to convey the separated storm water runoff to the Eastern and Western Ditches;
- closure of 12 out of the 15 existing CSOs and refurbishment of the remaining 3 CSOs; and
- renovation of substantial lengths of existing sewer to overcome identified structural deficiencies.

The proposed CSO closures eliminate all CSO discharges to the Western Ditch, but leave the three CSOs discharging to the Eastern Ditch and directly to the Ballinderry River. EHS specified that any CSOs discharging to the Eastern Ditch which were to be retained should incorporate storage sized in accordance with the recommendations of the SDD Guidelines [2]. Due to the low base flow in the ditch, this requirement led to the provision of a total of approximately 1200 m^3 of storage at two of the three remaining CSOs.

The existing WTP was constructed in 1965 and has become both hydraulically and biologically overloaded. The effluent is consistently of poor quality and fails to meet its registered standard conditions. The WTP is considered to be a major cause of the deterioration in river water quality from upstream to downstream of the Cookstown urban area. As a consequence of these considerations, the WTP is regarded as totally unsatisfactory and a new works is to be built on the site of the existing works. The registered standard for the new WTP was agreed between Water Service and EHS prior to the commencement of the UPM study.

The aim of this study was to evaluate the performance of the Cookstown wastewater system, incorporating the proposed improvements to both the sewer system and the new sewage treatment plant, against the environmental criteria specified by EHS. These were achieving both the Fundamental Intermittent Standards for a Sustainable Salmonid Fishery [5] and river chemical Class B, Good Quality. The methodology adopted for the study was specified by the EHS and was taken directly from the Urban Pollution Management Manual [5]. The key activities were:

Task 1: Construction of a simplified integrated model of the sewer system, sewage treatment works and receiving river incorporating the proposed improvements

This was achieved using SIMPOLv2 software. The closure of the majority of the existing CSOs allowed a simple model layout to be conceptualised. The bulk of the combined sewer system was represented in only two sub-catchments. The areas where storm flows are planned to be separated from the combined system were included as two additional sub-catchments with zero continuation flow and all storm flows discharged to the watercourses. The continuation flows come together at the CSO controlling inflow to the STW which is represented by a further sub-catchment, plus and a storm tanks module. The river system is represented in three reaches commencing at the point of input of the WTP effluent with reach divisions at the inflow of the Killymoon River and the Eastern Ditch. The downstream boundary of the study area was 10 km below the WTP input.

Task 2: Calibration of the SIMPOLv2 model

Hydraulic calibration of the sewer module was achieved using the results for a suite of 10 storm events from an existing HydroWorks model. Quality calibration was achieved using default values derived from previous UPM studies. Variation in the quantity and quality of base flows was included in the model following conventional diurnal patterns in 6-hour time intervals. Simulated concentrations at the bottom end of the sewer system were compared with the limited historicl quality data available at the WTP inlet and found to be a slight to moderate overestimate. This was deemed to be acceptable as it would result in a conservative assessment of the impact on the receiving water.

Task 3: Rainfall Time Series

A 10-year synthetic time series was generated using STORMPAC [5]. Calibration data were available in the form of a 15-year historical daily record from the Cookstown weather station. In addition, a set of 827 summer storms were extracted from the complete dataset to allow investigation of the critical conditions during the summer periods.

Task 4: Running the SIMPOL Model – Generation of Baseline Results

Baseline results, presented in Table 2, for assessing compliance with the FIS for a Sustainable Salmonid Fishery were generated by running the SIMPOLv2 model for the summer periods of the 10 years. This was achieved by dividing the 10 summers into 6,083 six-hour time intervals; 827 of these were the rainfall events. The remainder were assumed to be dry. This form of analysis allowed the incidence of failures during both dry weather periods and wet weather periods to be identified.

Tab. 2. Cookstown Baseline Results – Assessment Against Fundamental Intermittent Standards

Dry Weather	**DO**			**un NH3**		
525.6 six hr periods	**1 year**	**3 month**	**1 month**	**1 year**	**3 month**	**1 month**
Reach 1	0.0	0.0	0.0	0.2	0.3	1.6
Reach 2	0.0	0.6	1.6	0.0	0.1	0.3
Reach 3	0.0	0.2	1.0	0.0	0.1	0.5
Wet Weather	**DO**			**un NH3**		
82.7 six hr periods	**1 year**	**3 month**	**1 month**	**1 year**	**3 month**	**1 month**
Reach 1	0.0	0.0	0.0	0.3	0.5	1.9
Reach 2	0.0	0.0	0.2	0.1	0.2	0.9
Reach 3	0.0	0.0	0.2	0.1	0.3	1.0
Total	**DO**			**un NH3**		
608.3 six hr periods	**1 year**	**3 month**	**1 month**	**1 year**	**3 month**	**1 month**
Threshold (mg/l)	4.5	5.0	5.5	0.04	0.035	0.025
Permitted failures/yr	1	1.67	5	1	1.67	5
Reach 1	0.0	0.0	0.0	0.5	0.8	3.5
Reach 2	0.0	0.6	1.8	0.1	0.3	1.2
Reach 3	0.0	0.2	1.2	0.1	0.4	1.5

Fails standard (no failures found)

The conservative assumption was made that all storm events occurred when sewage base flows were at their largest and most concentrated, i.e. between 08.00 and 14.00 hours each day. The figures in the Table indicate the average number of 6-hour time blocks per summer for which the standards' threshold values were exceeded. Further results were also generated to test compliance with the 1-hour FIS. To test compliance against the GQA standards required consideration of the complete 10-year period, not just the summers.

Task 5: Sensitivity Testing

The robustness of the baseline results to changing modelling assumptions and input parameters was tested by an extensive programme of sensitivity testing. The following changes were investigated:

- increased sediment quantities in the sewers (to high end of recommended range);
- reduced erosion rates of sewer sediments (halved);
- allowance for additional surface water runoff from new developments;
- reduced in-river dissolved oxygen concentrations; and,
- reductions in the quality and increases in the quantity of treated effluent from the proposed sewage treatment works.

The baseline results for the 6-hour FIS showed no overall failures, suggesting robust compliance with standards for a Sustainable Salmonid Fishery following implementation of the proposed urban wastewater improvements. In general, the conditions in Reach 1 immediately downstream of the STW input were rather worse than the lower reaches where the diluting effect of the Killymoon River is present. Results for ammonia were distinctly worse than those for dissolved oxygen, suggesting that this is likely to be the controlling factor in the quality of the Ballinderry River. The results also showed that poor river quality is more likely during wet weather, but can still occur during dry periods. However, neither occur frequently enough to threaten overall compliance.

Threshold failures of the GQA Standards were more common and there was an overall marginal non-compliance in Reach 1 for BOD. However, the results also showed that failures are just as likely to occur in dry weather as during wet periods, suggesting that the problem, if it exists, is associated with the background conditions in the river and/or the quality of the WTP effluent. In this context, it should be noted that the WTP was modelled as operating at the upper limit of its registered standard conditions which was a conservative assumption. This phenomenon was investigated further during the sensitivity testing. In general terms, the results of the sensitivity testing programme showed that the principal conclusions drawn from the base line results were extremely robust. The only significant failures induced during the sensitivity testing occurred when parameters, such as the in-river mixed dissolved oxygen resulting from the mixing of the river with the CSO spills, were ascribed values that were beyond the bounds of what could be considered reasonable.

The major conclusion of the Cookstown UPM study was that the proposed improvements to the urban wastewater system of Cookstown will definitely support the achievement of the long term river quality objectives for the Ballinderry River in terms of both the FIS and GQA standards. In addition, the retention of the three CSOs towards the bottom end of the sewer system is acceptable in environmental terms provided that they are upgraded as proposed to incorporate storage.

5. General Conclusions

The West Belfast UPM study represents an excellent example of the use of advanced, integrated modelling tools to evaluate the environmental effects of proposed engineering improvements on a large and complex urban wastewater system. In addition to simulating the long-term performance of an extensive combined sewer system serving a population of some 200,000 and incorporating numerous CSOs, the impact of these discharges has been assessed on both inland and tidal receiving waters. The study presented a major challenge to the capabilities of the modelling tools. This has required a substantial amount of innovation and development. The successful outcome of the study, in terms of the ability to produce meaningful results for highly complex systems with limited available data, has proved both the flexibility and inherent strength of the UPM procedure and associated modelling tools.

The Cookstown UPM Study demonstrated that it is possible to carry out a study:

- to test compliance of urban wastewater upgrading proposals with a wide range of environmental standards at moderate cost;
- to undertake such studies using data that are commonly already available, avoiding the need for expensive field data collection exercises; and
- to generate reliable and robust results in which it is possible to have a high level of confidence.

The two case studies demonstrate how the application of the UPM procedure has evolved to reflect the constraints of short study timescales and reduced investigative costs while maintaining the need to deliver reliable, cost-effective solutions to complex CSO pollution problems.

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INCREASING KNOWLEDGE OF CSO TECHNOLOGIES

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1. Introduction

In the UK, the current and future focus on capital investment in wastewater treatment is based around improving the quality of effluent discharging into our coastal and inland waterways. In particular funding is targeted towards sanitary parameters at smaller works and at combined sewer overflows (CSO's) as these are major infrastructure components that influence downstream river and coastal water quality. However one of the major problems associated with the design of such structures is the prediction of stormwater discharge quality, in terms of aesthetics (in the UK regarded as solids larger than 6 mm in 2 directions), BOD, TSS and NH4-N. Such performance has to be assessed for a range of operating conditions and variable influent flow and loads. Traditionally the solution for the retention of BOD and solutes has been to incorporate storage solutions, many of which are extremely costly. Screens are commonly used to retain the aesthetics in the system and there have been major research initiatives to assess the performance of screens [1]. Many of these devices are new and novel and currently there are a number of research programmes that examine the in-situ performance of screens and their requirement for maintenance. The performance of a screen is a function of many parameters but key variables include the design flow rate and the number and nature of the aesthetic particles introduced to the screen. This paper concentrates on two aspects of this research into aesthetics: Firstly, the prediction of the quantities of aesthetic pollutants and secondly the performance of static screens to retain these pollutants.

2. Quantities of aesthetic pollution

A study has recently been completed [2] to gain a better understanding of the quantities of aesthetic solids by monitoring the aesthetic pollutant production at the input into sewer systems in 3 catchments of different socio-economic-ethnic characteristics (by questionnaire and survey) and subsequently by an in-sewer sampling programme to record the aesthetic pollutants at the downstream end of the same catchments. The socio-economic survey provided demographic data relevant to aesthetic pollutant production and seed factors were derived to quantify the amounts

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of different types of aesthetic solids – primarily sanitary solids and toilet tissue, from each of the catchment types.

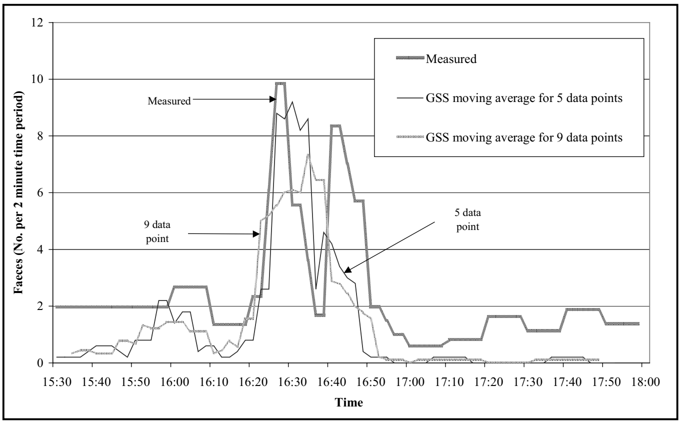

Figure 1. Comparison of predicted and measured aesthetic solids

The second phase of the work was concerned with a field evaluation of the gross solids characteristics within each of the three different catchments and the development of two component models, one to describe the transport in pipes less than 300 mm diameter and the second to describe the transformation of the aesthetic particles in the main carrier sewers of each catchment. A comparison of measured and simulated data is shown in Figure 1. A CSO model to determine the final destination of these solids was developed and the outputs from the CSO model provide the temporal change in gross solids that may be anticipated to be spilled during individual storm events. To retain these types of pollutants within the sewer system the UK water industry has adopted the wide scale use of screens in CSO chambers. The design guide of a small chamber specifically developed for a screened CSO may be found at www.wagug.org.uk.

3. Performance of static screens

In recent years there has been significant developments in CSO screening technology and in the UK a series of research studies and pilot evaluations have been completed, see FWR [3] and Thompson and Saul [1]. This paper describes the performance of a static screen under different flows and aesthetic loading conditions when placed in a single high side weir overflow chamber. The test rig was located at the UK National CSO Test Facility, Wigan Wastewater treatment works where live sewage is transferred from the inlet to the works into a specially constructed full-scale test facility. This inflow was seeded with aesthetic solids collected from the inlet screenings such that the concentration of aesthetic solids was up to 10 times (by weight) that observed in the dry weather flow to the works.

The test screen had a nominal length of three metres and a nominal width of one metre, giving a plan area of three square metres. The screen was constructed from panels of wedge-wire with 6mm apertures (diagonal) with each panel arranged to form a series of peaks and troughs at an internal angle of 60 degrees. The screen had 6 panels.

Tests were completed in 2 formats: Individual tests with a range of steady inflow rates and a range in the dose rate of aesthetics, and a continuous test where the inflow was in the form of a hydrograph with stepped increases and decreases of the inflow, again with different loading rates. Detailed results from the tests can be found elsewhere [1].

The results of these 2 tests highlight that with time the depth of flow in the chamber is increased as the test is progressed. This is due to the fact that the screen becomes blinded and an increase in head is required to drive through the flow through the screen. The rate of blinding was observed to be different at different flow rates and loading rates, illustrated by the change in gradient of the depth versus time plots. This information has been used to develop software to describe the blinding rate for this type of screen at a particular flow and loading rate. Hence given the outputs from the aesthetic production simulation software and a conventional hydraulic model it is feasible to predict the rate at which the screen will blind and to set alarm trigger levels for effective maintenance. These may be remotely sensed such that inspection time may be reduced to a minimum.

4. Finely Suspended Solids and Pollutants in Solution

In respect of the performance of chambers to retain pollutants in fine suspension and solution, for example BOD and ammonia, research has argued that storage facilities or high rate treatment options are required.

The concentration of these pollutants is known to vary with time and a typical trace is shown in Figure 2. BOD for example highlights a flush effect whereas ammonia illustrates a dilution effect.

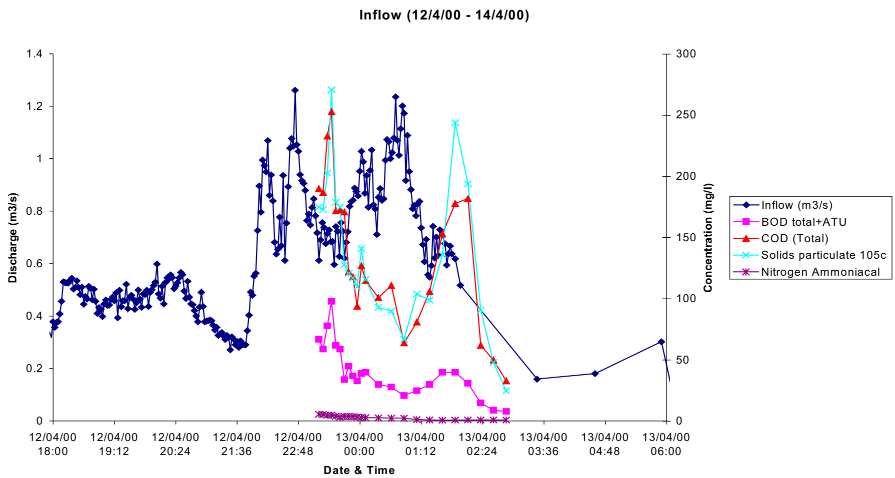

Figure 2. Temporal changes in flow and pollution

The conventional approach adopted in the UK to retain this type of pollution has been the inclusion of large and costly underground storage facilities, located at strategic locations throughout the catchment. As a consequence the cost effectiveness of alternative technologies has been questioned and a recent UKWIR research project has investigated the use of unconventional treatment options for CSO discharges as alternatives to sewer storage, Landon et al [4]. This research identified some 49 aerobic treatment processes that are currently used within the UK. These were classified into seven treatment categories: settlement, flotation, filtration, centrifugation, biological treatment, disinfection, and the physical/chemical removal of ammonia. Based on CAPEX, maintenance and operation, nuisance criteria, sensitivity to flow changes, footprint, power requirement and chemical usage, Landon et al [4] identified a short list of potential options of unconventional treatment for CSO spilled effluents. These were:

- **Ballasted sedimentation** - provides for a compact treatment process which occupies less area than chemically-assisted sedimentation;
- **Chemically-assisted lamella sedimentation –** particles are settled between thin plates between which the velocity of flow is low.
- **Lamella Dissolved Air Flotation (DAF)** – dissolved air is used to enhance the separation of particles such that the units have a smaller footprint compared to other types of DAF.
- **Pulsed-bed sand filters –** sand is used to filter the wastewater after settlement. These systems have the lowest head requirement of all the different types of sand filter.
- **Vortex separators** with chemical enhancement that enhance separation due to flocculation and dynamic separation.
- **High-rate biological filters** for the removal of particulate and soluble BOD and SS.

- **Rotating Biological Contactors** that are suitable for the removal of soluble BOD and also nitrification.
- **Submerged Aerated Filters and Biological Aerated Filters** are suitable for removal of soluble BOD and also nitrification.
- **Membrane activated sludge treatment** is compact and produces a high-quality effluent. Again there may be issues of ensuring that the membrane can discharge adequate flow for all expected inflow intensities, but this process may be worth further investigation. There are also requirements associated with ensuring adequate screening of the stormwater to ensure that the membranes do not rapidly foul which would make the process uneconomic.
- **Reed beds** have been used for treatment of stormwater at wastewater treatment works. The land area required by a reed bed is likely to make the process only feasible for CSOs with small spill volumes.
- **Chemical Disinfection (e.g. chlorination)** using sodium hypochlorite is a low-cost method for treating spills from CSOs. Peracetic acid and chlorine dioxide may also be suitable. By-products from chemical treatment may have a harmful effect on the environment and this approach is not accepted by the Environmental Regulator for continuous discharge.
- **UV disinfection** is the method recommended by the Environmental Regulator to disinfect secondary effluent from wastewater treatment works. Primary effluents contain higher levels of suspended solids, which shield micro-organisms from UV radiation. Hence pre-treatment of CSO spills would be required. Use of iron-based coagulants can decrease UV transmission because iron absorbs UV. In addition iron and aluminium salts have been implicated in fouling of UV systems.

The study concluded that there was no simple, cheap practicable solution to improve the quality of CSO discharges and that individual solutions would be site specific. They presented generic comments, requiring further verification, that suggested for BOD and SS removal, reed bed treatment systems may be appropriate for small rural catchments, ballasted lamella plate sedimentation systems appear most suitable for large urban CSOs and that for the oxidisation of ammonia suitable options would include biological aerated filters and membrane activated sludge plants.

This study has formed the basis of much further research in the UK the details of some of which will be presented at the workshop.

5. Concluding Remarks

There has been much research in the UK to assess the performance of sewer systems and of ways in which the pollution of inland and coastal waters may be protected from pollution. Some of the techniques may be applied to cold and temperate climates but others not. This paper sets out the approach in the UK.

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UPGRADING THE NORTH TORONTO COMBINED SEWER OVERFLOW (CSO) STORAGE AND TREATMENT FACILITY

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1. Introduction

Combined sewer overflows (CSOs) represent one of the major causes of deterioration of water quality in receiving waters and impairment of beneficial water uses [1]. CSO impacts are usually characterised with respect to their nature, time scales, spatial scales, and the types of receiving waters. The nature of the impact can be classified as physical, chemical, microbiological or combined [2]. Examples of physical impacts include increased flows, erosion and sediment transport/deposition alterations. Chemical impacts contribute to changes in water quality through dissolved oxygen depletion, nutrient enrichment and eutrophication, and possibly toxicity (both acute and chronic). Microbiological impacts often impair recreational waters (closure of recreational beaches) and shellfish harvesting areas. In specific receiving waters, these impacts occur in various combinations and are referred to as combined impacts, which can be measured by the biological community performance. Finally, another way to describe the CSO impacts is by ecological changes and impairments of beneficial water uses [3].

During the past 50 years, all new sewers in Canada have been built as separate sewers, but some older areas serviced by combined sewers can be found in all large Canadian cities. Consequently, the abatement of CSO pollution remains a priority in protection of water resources and all large Canadian cities with combined sewers have been implementing CSO abatement programs. The means of CSO abatement are rather varied and the choice of measures reflects local conditions. In general, these measures can be classified into four basic categories, (a) control of inflow of stormwater into combined sewers, (b) sewer separation (requiring many other measures as well), (c) CSO storage and treatment (essentially increasing the system collection efficiency and treating stored flows at central or peripheral water treatment plants), and (d) increased collection efficiency by improved system operation (e.g., real time control) [4].

The abatement of CSOs is governed by various federal or provincial regulations. In Ontario, the procedure F-5-5 is of particular interest and calls for the equivalent of

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primary treatment of CSOs before discharge into receiving waters [5]. With respect to total suspended solids (TSS), a 50% removal is required. Where CSO discharges occur upstream of recreational waters, disinfection is also required. The level of treatment required by F-5-5 cannot be achieved in conventional CSO storage tanks with relatively short settling times, but it could be attained by applying chemically aided settling in such tanks.

The paper that follows focuses on upgrading CSO control at the North Toronto CSO Storage and Treatment Facility (NTCSO Facility). It is an older CSO storage tank, which was modified for field experiments with chemically aided settling. This process represents an intermediate level of treatment meeting the provincial F-5-5 regulation requirements; higher efficiencies could be obtained with some proprietary processes, at higher costs. The main purpose of this paper is to present a CSO abatement measure, which can be implemented at a municipal level and serve for upgrading the performance of existing CSO storage tanks by retrofitting. The paper starts with a description of the facility, hydraulic analysis of facility capacity, chemically aided settling, and assessment of effluent quality.

2. North Toronto CSO Storage and Treatment Facility

The North Toronto Water Pollution Control Plant (NTWPCP) serves an urban area of about 3060 ha, of which two thirds are served by combined sewers and the rest by separate sewers. Land use in this area is mostly residential (78%), with some open space (13%) and industrial/commercial/institutional lands (9%). A controlled sewage flow of 0.4 m^3/s from the area is intercepted and treated at NTWPCP; dry weather flows in excess of this rate are diverted and treated elsewhere. In wet weather, flows in excess of 1.7 m^3/s overflow into a CSO storage facility with a capacity of 6000 m^3. The CSO storage facility, located by the NTWPCP, captures only a portion of the annual CSO volume from the sewershed studied and overflows to the Don River, about 30-40 times per year. CSO modelling for this sewershed indicated that a tank with a volume of 73,000m^3 would reduce the frequency of CSOs at this site to about one per year [6].

The layout of the North Toronto CSO Facility is shown in Fig. 1. In wet weather, CSOs from an adjacent combined sewer enter the facility inlet channel, continue through four connecting pipes into a distribution channel, and pass over inlet weirs into three storage tank cells, with a total volume of about 4,000 m^3. The inlet weir crest elevations are arranged stepwise at three levels, so that storage cells are filled sequentially, starting with the most downstream one. Overflows from the storage tanks are conveyed by the effluent channel into a stormwater tank (approximate volume of 2,000 m^3) where they mix with stormwater discharged from a storm sewer into the tank. When the stormwater tank is full, it overflows via the final effluent weir, and its effluent is blended with the secondary effluent from NTWPCP before being discharged into the Don River. It was observed that the pipes connecting the inlet and distribution channels limit the flow through the CSO storage tanks; at Q = 4.0 m^3/s, some flow starts to overflow from the inflow channel via a bypass weir directly into the stormwater tank. After storms, the contents of the North Toronto

CSO Facility (approximately 6,000 m^3) are pumped to the NTWPCP for treatment.

Thus, depending on the severity of the CSO event, the facility operates in two modes: (1) A batch mode, for small volume CSO events ($V < 6,000\ m^3$), which are fully contained in the storage facility, and eventually treated at the WPCP, and (2) a flow-through mode, for larger volume events ($V > 6000\ m^3$). Under such conditions, the facility overflows and these overflows receive limited treatment (i.e., by dynamic settling). Some portion of the total overflow is subject to settling in the CSO storage cells and the stormwater tank; for high inflows (> 4.0 m^3/s), some portion of flow bypasses the CSO storage cells and is subject to settling only in the stormwater tank with reduced settling times. The performance of the facility for CSO abatement can be improved in two ways, by increasing its hydraulic capacity (which is an effective measure only if combined with better treatment) and by improving the CSO treatment efficiency.

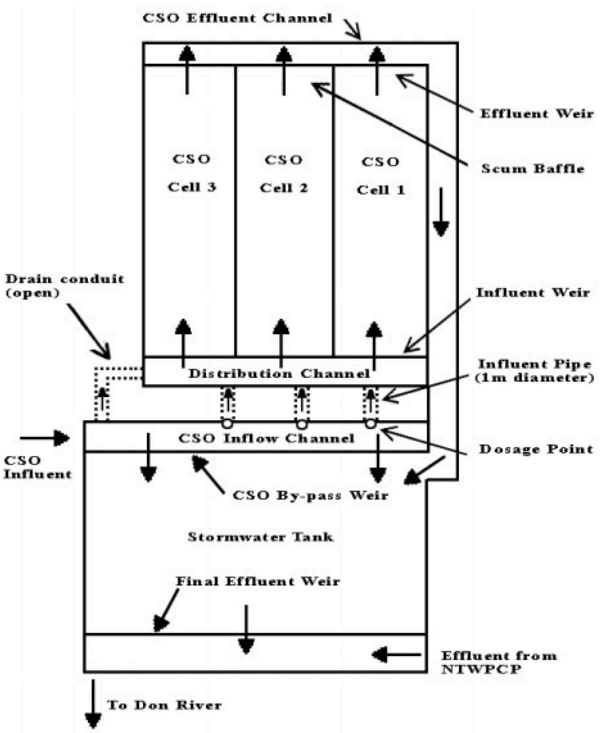

Fig.1. North Toronto CSO Storage and Treatment Facility

3. Improving the North Toronto CSO Storage and Treatment Facility Performance

During the last four years, the North Toronto CSO Facility has been studied with the overall objective of improving the facility performance in the abatement of CSO pollution. These investigations focused on two aspects, improving the facility hydraulic efficiency and implementing chemically aided settling of overflows by addition of coagulants and/or flocculants inducing faster settling of suspended solids in CSOs passing through the facility. The treatment efficiency of this process depends on the treated medium properties, chemical addition, and favourable hydraulic conditions, which would induce particle coalescence in the coagulation zone and quiescent settling downstream of this zone. This treatment process would greatly benefit from favourable hydraulic conditions in the cells, with uniform flow distribution and low turbulence. These issues are addressed in the following sections.

3.1. IMPROVING HYDRAULIC EFFICIENCY OF THE FACILITY

As stated in the previous section, the inlet channel starts to overflow directly into the stormwater tank when inflow reaches a discharge of 4.0 m^3/s. Further increases in the inflow will mostly overflow into the stormwater tank, and the rest will follow a longer flow route by entering the CSO storage cells (with proposed chemically aided settling), and passing though the effluent channel into the stormwater tank. The feasibility of increasing the treatment capacity of the CSO cells, without any overflows from the inlet channel, was addressed by numerical and physical modelling. In particular, it was of interest to assess some structural changes in the facility layout, which would increase the flow through the CSO storage cells. Two basic options were considered – (a) reducing the final effluent weir height (which controls the outflow from the stormwater tank), and (b) improving hydraulic efficiency along the entire flow path from the CSO storage cells to the final effluent weir. It should be noted that reductions in the final effluent weir height will reduce storage (2,000 m^3) in the stormwater tank, and therefore reduce the corresponding settling time. This may adversely affect the settling of direct overflows from the inflow channel (the effectiveness of this settling also depends on flow conditions in the tank), but affect less significantly the final settling of flows subject to chemically aided settling in the CSO storage cells.

The hydraulics of the CSO facility was investigated by two methods applied conjunctively, a physical scale model and a Computational Fluid Dynamics (CFD) model. Further details follow.

The physical scale model was constructed in the scale of 1:11.6 and designed according to the Froude similitude. The model was instrumented by pressure sensors for measuring water depths and an ADV velocity probe was used for accurate flow velocity measurements in three directions. Furthermore, four Pitot tubes were used to measure velocities in the four pipes interconnecting the inflow and distribution channels. The discharge through the facility was measured by a V-notch weir [7].

The physical model was found to be well suited for addressing all hydraulic issues concerning this CSO facility. It provided good results for flow capacities reflecting various structural changes, which were easily implemented in the model of this size (a plywood box 3 x 7 x 0.6 m, with partitions). Besides providing the values of various hydraulic parameters, the model also served to verify the CFD model. Some results regarding the facility flow capacities and the effects of reducing the final effluent weir are presented in Table 1 [7].

A number of inferences can be drawn from the data in Table 1 – the benefits obtained by reducing the final effluent weir are limited and diminish with greater reductions in weir height. The first reduction step (by 0.2 m) is most effective. It would increase the cell flow capacity by 11% without any other changes, and by almost 19%, if the bends in the effluent channel were also hydraulically improved. This measure would reduce the stormwater tank storage for zero flow conditions by about 13%. A complete weir removal would increase the CSO cells capacity by 34%, but eliminate the stormwater tank and its storage completely. For higher discharges through the CSO cells, the corresponding settling times will be shorter.

Table 1. CSO Facility flow capacity, without overflows of the inlet channel, measured in the physical scale model [7]

Reduction in the height of the final effluent weir [m]	Max. flow capacity without inlet channel overflows (no other structural changes) [m^3/s]	Max. flow capacity, without overflows, after changing both effluent channel bends [m^3/s]
0.0	4.08	4.38
0.2	4.52	4.84
0.6	5.00	5.14
1.5 (weir removed)	5.46	(not measured)

CFD modelling was conducted with PHOENICS software [8], recognizing that other competing products may be equally well applicable [9]. Computer runs were done using the multi-phase model with a structure mesh, also referred to as the "algebraic slip" method. Two turbulence models were tested in the study, the level turbulence model and the well-known k-ε model. The latter model was selected for further work, recognising that it was not ideally suited for this application with high flow complexity. The numerical accuracy of CFD flow simulations increases with the number of cells in the grid, but so does the computation time. For complex structures, like the CSO facility studied, a compromise had to be found between the number of cells and the computer running time. The choice of modelling grid is also important for numerical modelling; in this study, 55 x 45 x 25 uneven fixed nodes were used in X, Y and Z directions, respectively, with a time step of 0.182 second and 15 iteration sweeps [7].

The CFD model was verified in two ways, by checking mass conservation in numerical runs and by comparing simulated results to observations in the physical scale model. In the first method of verification, mass conservation in various runs was preserved within ±1%. In verifications against the scale model, water levels and their changes, at various locations within the facility, were compared. Simulated and measured water surface profiles agreed well, as shown for a typical case (the most downstream storage cell #3) in Fig. 2 [7].

3.2. FLOW CONDITIONING IN STORAGE CELLS

The CFD model was also used to establish flow velocity distribution in the CSO storage cells. A highly turbulent flow in the distribution channel enters the CSO cells with a non-uniform distribution of velocity across the cell entrances, and may cause strong downward and rotational velocities in the CSO cells. These flow features produce velocity shear, flow turbulence and strong bottom shear stress in cells impairing pollutant settling and potentially causing the scouring of deposited sludge. Consequently, numerical simulations were carried out to design flow conditioning baffles which would remedy this situation. A preliminary design of flow conditioning baffles is shown in Fig. 3. Flow velocity distributions with and without inlet baffles are shown in Fig. 4. The displayed results indicate marked improvement in flow patterns, which should facilitate more effective settling.

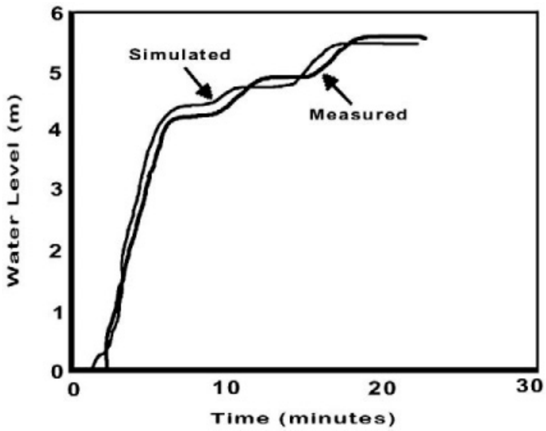

Fig. 2. Measured and simulated variations in water levels in Storage Cell 3 [6]

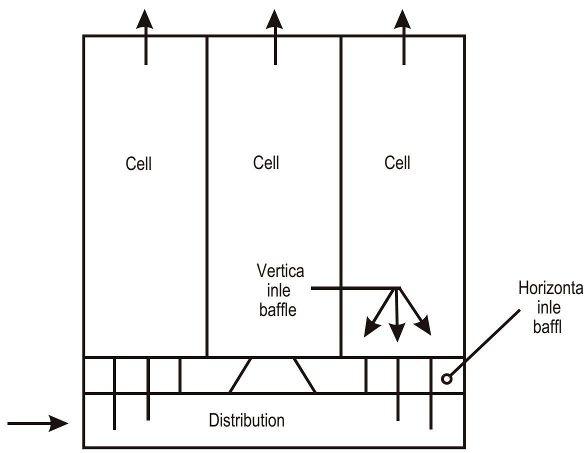

Fig. 3. Proposed inlet baffles providing more uniform flow distribution

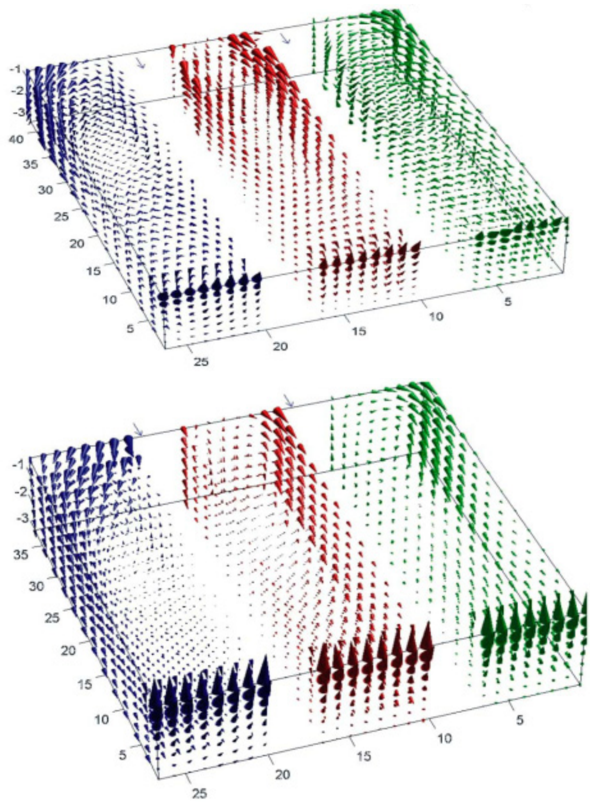

Fig. 4. Flow distribution in storage cells with (bottom section) and without (top section) inlet baffles [7]

3.3. CHEMICALLY AIDED SETTLING

In designing chemically aided settling, the important points include (a) wastewater characteristics (settleability), (b) choice of coagulant/flocculant, (c) method of dosing, and (d) the overall system effectiveness (quality of the treated effluent) [10, 11]. Further discussion of these points follows.

Wastewater settleability is typically assessed by long-column testing, which is well described in various handbooks [12]. Settling column testing has one limitation, it does not reproduce well the actual field conditions in settling tanks, characterized by flow through and some turbulence. Consequently, the authors have been working on developing an elutriation apparatus for testing settling [13]. In this apparatus, flow is pumped from the downstream end through a series of settling columns, fed with the test medium and arranged in such a way that the flow is rising through these columns. Starting from the largest diameter cylinder, the cylinder diameters are reduced by factor of two in the upstream direction, and the flow velocities increase four times. Particles with settling velocities lower than the rising velocity get washed out into the next column(s), until they reach a column with a lower rising velocity and settle. Thus, this apparatus provides an assessment of settleability under dynamic conditions and mimics field conditions better than settling columns. An example of settleability tests performed on a CSO sample is shown in Fig. 5 for two samples, with and without a flocculant (polymer) added.

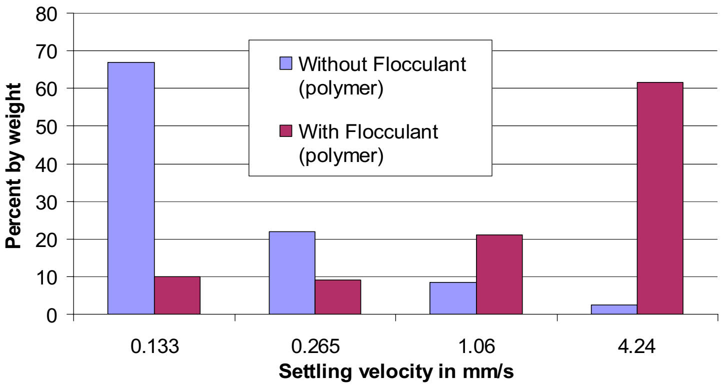

Fig. 5. Elutriation testing of CSO settleability with and without polymer addition

Many coagulants and flocculants are available on the market to enhance wastewater treatment through coagulation and flocculation. In general, these chemicals fall into two groups, inorganic salts and polymers. Each group has some pros and cons; salts are generally cheaper and relatively less toxic, but sensitive to pH variations, require high dosages and may produce more sludge. Also, coagulants containing iron may not be suitable in installations where UV disinfection is contemplated. Polymers are fairly effective at low dosages and do not change pH of the wastewater, but are more expensive and may cause toxic effects even at relatively low concentrations [14]. Jar tests conducted with a synthetic wastewater may help in systematically evaluating the influence of individual parameters (such as suspended solids, dissolved organics, or pH fluctuations) on the effectiveness of a given coagulant or flocculant, aiding in the choice of a robust treatment scheme. Once a suitable coagulant, flocculant, or combination has been chosen, further jar testing with the wastewater to be treated is helpful in determining the appropriate dosage range. An example of test results conducted with a synthetic wastewater (mimicking mainly suspended solids and dissolved organic matter) is shown in Fig. 6.

Two methods of flocculant/coagulant dosing are commonly used – flow proportional and solids mass flux proportional dosing. The first case is relatively straightforward and requires only flow measurement data to calculate the rate of dosing of the coagulant/flocculant to maintain its constant concentration in the wastewater treated. However, this method of dosing suffers from some limitations. In particular, the chemical is dosed even when not needed, e.g. when the suspended solids concentrations fall below a permissible value, and this not only contributes to higher operating costs, but may also increase the risk of toxicity of the treated effluent [15]. Polymer, which is not utilized (bound) by solids, may attach to fish soft tissues and cause toxic effects [14]. The other common dosing method is based on solids mass flux. This case requires more complex instrumentation, typically encompassing a flow meter and a turbidity meter, which serve to estimate the solids flux. The flocculant is then dosed proportionately to this flux.

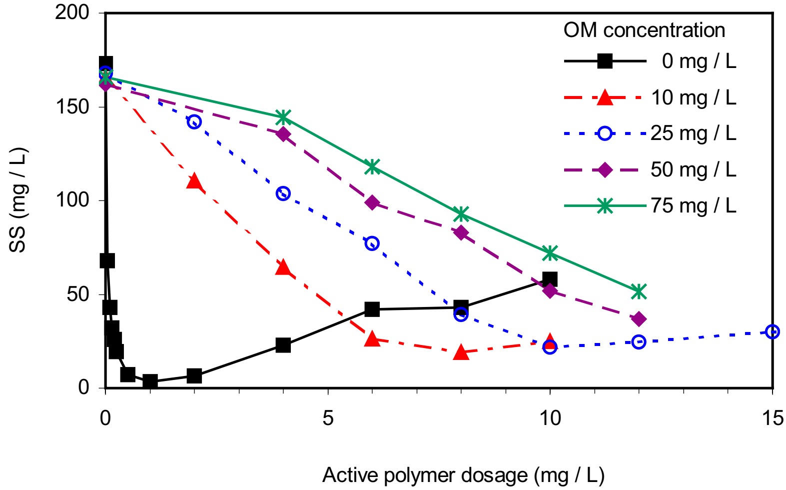

Fig. 6. Suspended solids removal using a polymer flocculant at varied initial organic matter (OM) concentration

The effectiveness of chemically aided settling has been reported in the literature for various conditions and types of wastewater. In the earlier studies in Ontario with flow-proportional dosing, the effectiveness assessment focused on removals of suspended solids, which are emphasized in the Policy regulation F-5-5 [5]. In the settling of CSOs, the desired removals of 50% were obtained at unsteady surface loads ranging up to 30-40 m/h, however, with large uncertainties in these removals [6]. More recent experiments focused on steady surface loads of 15 m/h, and produced the following removals at the North Toronto site: 75% of TSS, 53% $CBOD_5$, 52% COD, 22% TKN, 43% Cr, 60% Cu and 51% Zn, for a polymer dosage of 8 mg/L. Similar results were produced for stormwater (from separate storm sewers) as shown in Table 2. The surface loads employed were limited by the equipment or facility configurations.

In conjunction with these tests, the acute toxicity of treated effluent was assessed at the North Toronto CSO facility using a 96 h rainbow trout bioassay. Non-toxic effluent was confirmed in 17 out of 19 cases (events) studied, in one case, the influent was toxic but effluent non-toxic, and in the last case, both influent and effluent were toxic. Although no increase in toxicity due to polymer addition was noted, accidental polymer overdosing has to be prevented by control algorithms.

The implementation of solids-mass-flux proportional dosing is under way at the facility studied. It was delayed by the need to develop a reliable and robust relationship between turbidity readings and suspended solids concentrations.

Table 2. Suspended solids removal rates from stormwater and combined sewer overflow high-rate treatment sites

Location	Design Polymer Dosage [mg/L]	Surface Loading Rate [m/h]	TSS Removal [%]
Etobicoke High-Rate Stormwater Treatment Site	4	15	84
North Toronto CSO Storage and Treatment Facility	8	15	75

4. Conclusions

Traditional CSO storage facilities can be environmentally upgraded by optimising their hydraulics and implementing chemically aided settling. In the case study presented for the North Toronto CSO facility, a number of structural measures serving to enhance the facility treatment rate and inducing favourable settling conditions in the settling basins were addressed by means of physical and computer modelling. A physical scale model was effective for establishing the hydraulic performance of the facility (flow rates, water levels, and changes in these parameters) and verifying a CFD model. The CFD model simulated well the hydraulic phenomena in the facility and will be used in the next study stage for particle tracking. With respect to chemically aided settling, the settleability of CSOs can be assessed well by elutriation testing. The choice of a coagulant/flocculant depends on the characteristics of the wastewater treated, including pH, dissolved organic matter contents and characteristics, and temperature. Jar testing offers the best way for selecting the most suitable chemical and its dosage. Finally, the solids removals in excess of those required by a provincial (Ontario) CSO control directive (primary treatment equivalency) were achieved at the facility studied with flow-proportional polymer dosing of 6-8 mg/L and the surface load rates of 15 m/h. Further refinements of settling conditions in the facility (by flow conditioning baffles), dosing (switching to solids flux proportional dosing) and higher surface loading rates are planned and will be assessed by both CFD modelling and field observations.

5. Acknowledgements

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IMPROVING COMBINED SEWER OVERFLOW AND TREATMENT PLANT PERFORMANCE BY REAL-TIME CONTROL OPERATION

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1. Introduction

In recent years, urban stormwater discharges have been recognised as one of the main causes of the environmental problems of receiving water bodies. During rain events, in fact, large amounts of water and pollutants are directly discharged through combined sewer overflow (CSO) devices into the receiving waters without any treatment [1, 2, 3, 4]. In addition, storm flow peaks and pollutant shock loads can arrive at the treatment plant (TP) reducing its efficiency or causing malfunctioning.

Recently, environmental problems have induced researchers to focus on the qualitative impacts of the CSO discharges. In particular, more efficient CSO devices, such as the high crested side-weir with bottom orifice and many kinds of vortex separator devices, have been proposed by different authors. These devices can achieve a relevant reduction of the discharged pollutant loads, besides the usual flow partition function [5, 6, 7, 8].

A further significant improvement of the performance of CSO devices can be obtained by increasing as much as possible the storage capacity of the sewer system. For this objective, the real-time control (RTC) has been proven to be a cost-effective solution since it is based on the activation of existing unused capacities [9, 10] and consequently allows for reducing the need of new storage infrastructures. Schilling [11] reports significant monetary savings achieved by the implementation of RTC for several case studies. Benefits obtained with the implementation of RTC have been shown by different authors [12, 13, 14] by comparing the performance of various urban drainage systems in RTC scenarios and in uncontrolled system scenarios.

One common RTC technique is the activation of the sewer in-line storage capacity by placing moveable regulators, such as sluice gates, weirs and orifices, into the main trunk sewers. These regulators, if adequately controlled, can reduce the CSO discharges in terms of both water volumes and pollutant masses and can provide also a better regulation of the flows conveyed to the TP.

In this paper, the improvement of the performance of a CSO device derived from the adoption of RTC techniques is analysed in terms of reduction of discharged water volumes and pollutants. In particular, a simple quality-oriented CSO device, the high crested side-weir with bottom orifice, has been considered. This device is made up of a high crest side weir and of a downstream fixed gate with bottom orifice and allows

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storing some in-line volume behind the gate until the water level rises up over the weir crest. The storage process leads to reduction of the discharge of first flush waters and of pollutants into the receiving water body and to an increase of the water volumes conveyed to the treatment plant [8].

Benefits obtained with RTC techniques have been evaluated considering moveable sluice gate regulators placed upstream of the CSO device. The control of the gates has been performed adopting a basic local strategy aiming to activate the maximum in-line storage. In particular, standard proportional (P) control units have been considered for the regulation of the gate movements [15, 16].

The analysis has been carried out by setting up and applying a numerical model to the experimental data of two Italian urban catchments, considering both quantitative and qualitative measurements recorded during several rain events.

The implemented model is based on the fully-dynamic St. Venant equations for the flow and on the advection-dispersion equation for the analysis of the pollutants in the sewer system.

2. Mathematical model

For a prismatic collector, the 1-D St. Venant equations for the unsteady flow and the advection-dispersion equation for the pollutant transport can be written in the conservative-law form:

$$\frac{\partial \mathbf{U}}{\partial t} + \frac{\partial \mathbf{F}(\mathbf{U})}{\partial x} = \mathbf{D}(\mathbf{U}) \tag{1}$$

where the terms $\mathbf{U}$ (dependent variable vector), $\mathbf{F}(\mathbf{U})$ (flux vector) and $\mathbf{D}(\mathbf{U})$ (source-diffusive term vector) can be written as follows:

$$\mathbf{U} = \begin{pmatrix} A \\ Q \\ AC \end{pmatrix}; \qquad \mathbf{F}(\mathbf{U}) = \begin{pmatrix} Q \\ VQ + F_h / \rho \\ QC \end{pmatrix}; \qquad \mathbf{D}(\mathbf{U}) = \begin{pmatrix} -q_o \\ gA(i-J) - Vq_o \\ \dfrac{\partial}{\partial x}\left(AD \dfrac{\partial C}{\partial x} \right) - Cq_o \end{pmatrix} \tag{2}$$

where x [m] and t [s] are space and time independent variables; A [m^2] is the water cross-section; Q [m^3/s] and V [m/s] are the flow rate and velocity respectively; F_h [N] is the hydrostatic force over the cross-section; ρ [kg/m^3] is the water density; g [m/s^2] is the gravity acceleration; i is the bed slope; J is the friction slope; q_o [m^2/s] is the lateral outflow discharge per unit length of collector; C [kg/m^3] is the pollutant mass concentration and D [m^2/s] is the dispersion coefficient.

The friction slope has been evaluated using the Strickler relation:

$$J = \frac{V^2}{k^2 R^{4/3}} \tag{3}$$

where R [m] and k [m$^{1/3}$/s] is the hydraulic radius and the collector roughness coefficient, respectively.

The lateral outflow discharge from the side-weir has been evaluated using the following relation:

$$q_o = \mu\sqrt{2g}\,(h-b)^{3/2} \tag{4}$$

where h [m] is the water level, b [m] is the height of the side-weir crest and μ is the discharge coefficient for the side-weir assumed to be equal to 0.38, as usual.

For both the fixed gate orifice of the CSO device and the real-time controlled gate openings, the following outflow relation has been used:

$$Q = C_c\, a\, B\sqrt{2g\left(h_{up} + \frac{V_{up}^2}{2g} - h_{dw}\right)} \tag{5}$$

where a [m] is the gate opening; B [m] is the gate width; h_{up} [m] and h_{dw} [m] are the water levels upstream and downstream of the gate; and, V_{up} [m/s] is the upstream flow velocity. The contraction coefficient C_c is derived from Ghetti [17] as:

$$C_c = 0.61 + 0.39\left(\frac{a}{h_{up}}\right)^{4.84} \tag{6}$$

The control of the gates can be performed by means of standard proportional-integral-derivative (PID) control units. On the basis of local water level measurements, these units calculate the deviation $e(t)$ of the water level h_{up} from its set point and determine the necessary gate movements $\Delta a(t)$ as [18]:

$$\Delta a(t) = K_p\, e(t) + K_i \int_0^T e(t)\,dt + K_d \frac{de(t)}{dt} \tag{7}$$

where the three terms involving $e(t)$ represent, respectively, the proportional, integral and derivative components of the correction. For an efficient control of different processes the PID units have to be calibrated by assigning adequate values to the parameters K_p, K_i and K_d. Basically, the proportional parameter K_p provides the velocity of the corrective action, the integral parameter K_i allows for addressing the process to the set point while the derivative parameter K_d allows for reducing the water level oscillations. In many cases the control of a process does not require the use of all the components and simpler units such as proportional *P*, integral *I*, proportional-integral *PI* and proportional-derivative *PD* controllers may be used. Additionally, for the described control units, the time interval between two successive control operations has to be set according to the desired control efficiency.

The pollutant dispersion coefficient D has been calculated using the following relation, theoretically derived by Elder [19] for turbulent shear flow conditions:

$$D = 5.93hu^{*} \tag{8}$$

where $u^{*}\,[\text{m/s}] = \sqrt{gRJ}$ is the friction velocity. For sewer systems, equation (8) generally provides values of D within the range 0.5-2.5, according to the values suggested by many authors [20, 21, 22, 23] for the propagation of pollutants in sewer collectors.

Considering low concentrations, water flow is assumed not to be influenced by the pollutant propagation phenomena. Therefore, equation (1) can be solved following a totally uncoupled approach [24]. In particular, the numerical solution of the St. Venant equations allows for determining the hydraulic variables used for the resolution of the advection-dispersion equation at the same time-step.

The second order predictor-corrector McCormack numerical scheme [25] was adopted for solving equation (1); an additional step based on the theory of the Total Variation Diminishing (TVD) [26] has been applied for reducing the numerical oscillations occurring for high gradients of water levels and flow rates.

Boundary conditions are imposed on the hydraulic variables Q and A at the upstream and downstream collector ends. In particular, an inflow hydrograph $Q(t)$ is considered at the upstream boundary and a relation $Q(h)$ is prescribed at the downstream end depending on the local flow conditions. Water cross-sections A are correspondingly evaluated by applying the method of characteristics [25]. The inflow pollutograph $C(t)$ is also imposed at the upstream collector end, as boundary condition for the advection-dispersion equation.

Finally, additional boundary conditions are prescribed with respect to the gates. These devices are treated as internal boundaries by providing both the variables Q and A upstream and downstream each gate [26]. In particular the flow rate value upstream the gate is derived from equation (5) while the continuity equation is used for deriving the flow rates downstream of the gate. Corresponding values of water cross-sections upstream and downstream of the gate are evaluated by the applying the method of characteristics.

3. The systems studied

The urban catchments of Cascina Scala (Pavia) and Fossolo (Bologna) have been considered in the present analysis.

Cascina Scala [27] is a residential district located on the northern outskirts of Pavia, Italy. The total contributing area is 11.35 ha (65% impervious, 35% pervious and not connected to the sewer system) and the number of inhabitants is about 3,000. An average flow rate Q_d of 0.007 m^3/s is conveyed in the sewer system during dry periods [28]. The final collector of the sewer system is characterised by a 70x105 cm egg-shaped cross-section and an average slope of about 0.45% [29]. A simpler 60x100 cm rectangular cross-section with a wetted area equivalent to the actual one (for the degree of filling of 80%) and a total length of 500 m have been considered for the collector in the numerical simulations. Assuming a Strickler coefficient $k = 70$ m$^{1/3}$/s, a maximum flow capacity Q_{max} of about 0.85 m^3/s has been obtained for the simulated collector.

Fossolo [30] is a residential area in the Bologna suburbs. The total area is about 40.7 ha (75% impervious, 12.5% pervious and directly connected to the sewer system and 12.5% pervious unconnected). Resident population is about 10,000 inhabitants. An average flow rate Q_d of 0.023 m^3/s was estimated for dry period. The outlet collector of the drainage system has a 180x144 cm polycentric cross-section and an average slope of 0.3%. Similarly to the previous case, a simpler 150x140 cm rectangular cross-section of a total length of 400 m has been considered for the collector in the numerical simulations. Considering a Strickler coefficient $k = 60$ m$^{1/3}$/s, a maximum flow capacity Q_{max} of about 3.20 m^3/s has been obtained for the collector.

A simple system (Figure 1) consisting of a long sewer collector (followed by the treatment plant) and of a high crested side-weir with bottom orifice (close to the downstream collector end) has been considered for the simulations in both catchments.

Moveable sluice gates have been considered upstream of the CSO device and controlled in real time with the specific objective to activate the maximum in-line storage capacity. Measured flow rates and pollutant concentrations have been assumed to enter at the collector's upstream end and to propagate downstream to the CSO device.

The high crested side-weir with bottom orifice has been considered to be placed 25 m from the downstream end of the collector in both the cases. The geometry of the high crested CSO device has been fixed according to the common design criteria for CSO devices in Italy. In particular, the bottom orifice of the fixed gate is designed with reference to a dilution threshold value of flow rate $Q_{thres} = r\,Q_d$, where r is a dilution coefficient, to be set conveniently within the range from 3 to 6. Obviously, the adoption of lower values of r leads to an increase in CSO discharges and a consequent decrease of treated volumes. In particular, the extreme values $r = 3$ and $r = 6$ have been considered in the present work. The weir crest level is sized in steady flow conditions equal to the critical depth h_{crit} relative to Q_{max}, while the side weir length is calculated as a function of Q_{max} and of the maximum treatment flow capacity $Q_{treatmax}$.

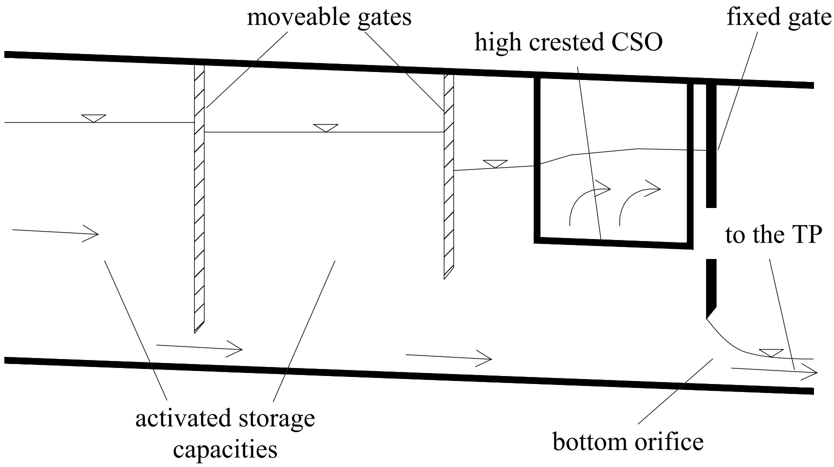

Figure 1. Simulated system for the two urban catchments.

The hydraulic and geometrical characteristics of the simulated CSO devices are listed in Table 1 and Table 2.

Table 1. Cascina Scala catchment: characteristics of the high crested CSO device.

	Q_{thres} (m^3/s)	$Q_{treatmax}$ (m^3/s)	Crest height (m)	Weir length (m)	Orifice opening (m)
$r = 3$	0.021	0.091	0.60	5.0	0.06
$r = 6$	0.042	0.134	0.60	5.0	0.09

Table 2. Fossolo catchment: characteristics of the high crested CSO device.

	Q_{thres} (m^3/s)	$Q_{treatmax}$ (m^3/s)	Crest height (m)	Weir length (m)	Orifice opening (m)
$r = 3$	0.069	0.347	0.80	12.0	0.08
$r = 6$	0.138	0.514	0.80	12.0	0.12

According to a procedure based on the activation of the maximum in-line storage capacity [13], two moveable sluice gates have been considered for both cases: at sections 240 m and 420 m from the upstream entrance of the Cascina Scala collector and at sections 160 m and 320 m from the upstream entrance of the Fossolo collector.

The RTC of the gates has been performed by adopting a local control strategy based on activating the in-line storage capacity and on maintaining a pre-determined set point water level (fixed equal to 80% of the collector height) behind each gate.

For this purpose, simple P units have been adopted for the control of the gates, closing or opening them every time the upstream water level is too low or too high, respectively, compared with the set point. Recent studies have shown in fact that, if adequately calibrated, the P control units can provide accurate water level regulations in sewer collectors [16]. According to the adopted local strategy, the movements of the two gates are independent of each other.

The gate control begins with a start-up phase which operates the closure from the initial opening condition allowing for quick in-line filling process. As the water level behind the gates reaches the set point, the start-up phase ends and the P units start to operate the regulators. According to the usual practice, in order to avoid sedimentation problems behind the gate [31], a minimum gate opening (equal to the height of the CSO bottom orifice) has been considered.

4. Experimental data

Experimental hydrographs and pollutographs for several rain events recorded in the two catchments have been considered in the present work. Among the different measured pollutants, suspended solids (SS) have been selected for simulations. Tables 3 and 4 show some characteristics of the analysed events for Cascina Scala and Fossolo catchments, respectively. As examples of the available data, Figure 2 shows the hydrographs $Q(t)$ and pollutographs $C_{SS}(t)$ of the suspended solids for the event No. 2 recorded in Cascina Scala and the event No. 1 recorded in Fossolo.

Table 3. Cascina Scala catchment: characteristics of the simulated events.

N	Date	Rain depth (mm)	Event duration (min)	Max. flow rate (m^3/s)	Max. SS concentr. (kg/ m^3)	Water volume (m^3)	SS mass (kg)
1	23/6/00	16.4	96	0.551	0.89	765	292
2	8/7/00	7.0	29	0.325	2.96	237	420
3	10/7/00	11.0	69	0.376	1.00	461	133
4	17/3/01	26.2	426	0.281	1.28	1445	196
5	28/3/01	18.6	437	0.263	2.36	985	205
6	10/4/01	8.4	108	0.157	1.20	450	191
7	20/4/01	15.8	368	0.257	1.19	910	340

Table 4. Fossolo catchment: characteristics of the simulated events.

N	Date	Rain depth (mm)	Event duration (min)	Max. flow rate (m^3/s)	Max. SS concentr. (kg/m^3)	Water volume (m^3)	SS mass (kg)
1	25/4/94	7.8	77	0.889	0.91	1402	563
2	21/9/94	11.5	184	0.490	0.14	2482	203
3	28/10/94	39.5	290	2.332	0.28	5255	607
4	20/2/96	14.8	114	0.439	0.89	1754	729
5	2/4/96	14.2	169	0.412	0.31	1861	594
6	21/8/97	9.8	46	1.018	1.49	1161	405

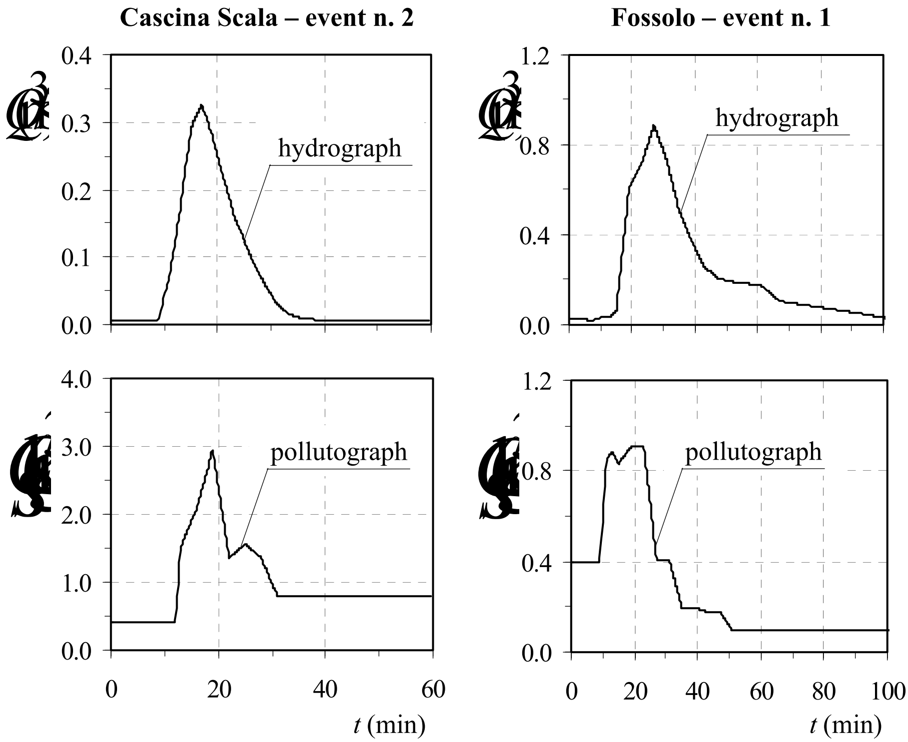

Figure 2. Hydrographs and pollutograph recorded in Cascina Scala (event No. 2) and Fossolo (event No. 1).

5. Simulations and results

All the events reported in Tables 3 and 4 have been simulated with the adopted model considering three different scenarios. Firstly the simulations for the no-control scenario have been carried out; then, the benefits of the introduction of the RTC have been investigated simulating a scenario with one moveable gate only (the one closer to the CSO device) and subsequently, a scenario with two moveable gates.

The gate speed has been fixed equal to 0.005 m/s, according to the usual values adopted in practical applications.

A value of $K_p = 1$ has been adopted for the P unit calibration in order to reduce the maximum deviation, achieve quick system reactions and reduce the occurrence of permanent oscillations [16].

Furthermore, the time interval between two successive control operations has been fixed to half a minute. This value insures the necessary control frequency to achieve good regulation efficiency.

For all the simulations a spatial step $\Delta x = 0.50$ m has been considered, while time steps Δt have been evaluated for the Courant number $C = 0.8$.

Simulations served to compute the hydrographs and pollutographs of the overflow discharges and of the flows to the TP. As an example, Figure 3 shows a comparison of the results obtained for the no-control scenario (only the CSO device) and for the RTC scenarios with one and two moveable gates. In particular, the graphs of the left side of figure 3 show inflow rates, the CSO flow discharges and the flow rates conveyed to the treatment plant for the event No. 2 in Cascina Scala and for $r = 6$.

Similarly the graphs on the right side of Figure 3 show the analogous results for event No. 1 in Fossolo and for $r = 6$.

In spite of the good performance of the adopted CSO device, both figures show remarkable advantages obtained by the introduction of the RTC. In particular, although flow rate peaks of CSO discharges are often slightly accentuated by the RTC (due to the simple local strategy adopted), a significant reduction of the total discharged water volumes with one moveable sluice gate and with the additional second gate is obtained. Moreover, the graphs point out that the RTC techniques can significantly increase the total treated volume allowing for conveying to the TP flow rates not exceeding the design maximum treatment capacity $Q_{treatmax}$.

A similar comparison concerning the suspended solid mass rates Q_{SS} entering the collector, Q_{SS} discharged through the CSO device and Q_{SS} conveyed to the treatment plant is reported in Figure 4. In particular, for the same events and conditions displayed in Figure 3, Figure 4 shows reduction of the discharged mass and the increase of the treated mass that is obtained with the adoption of one and two moveable gates.

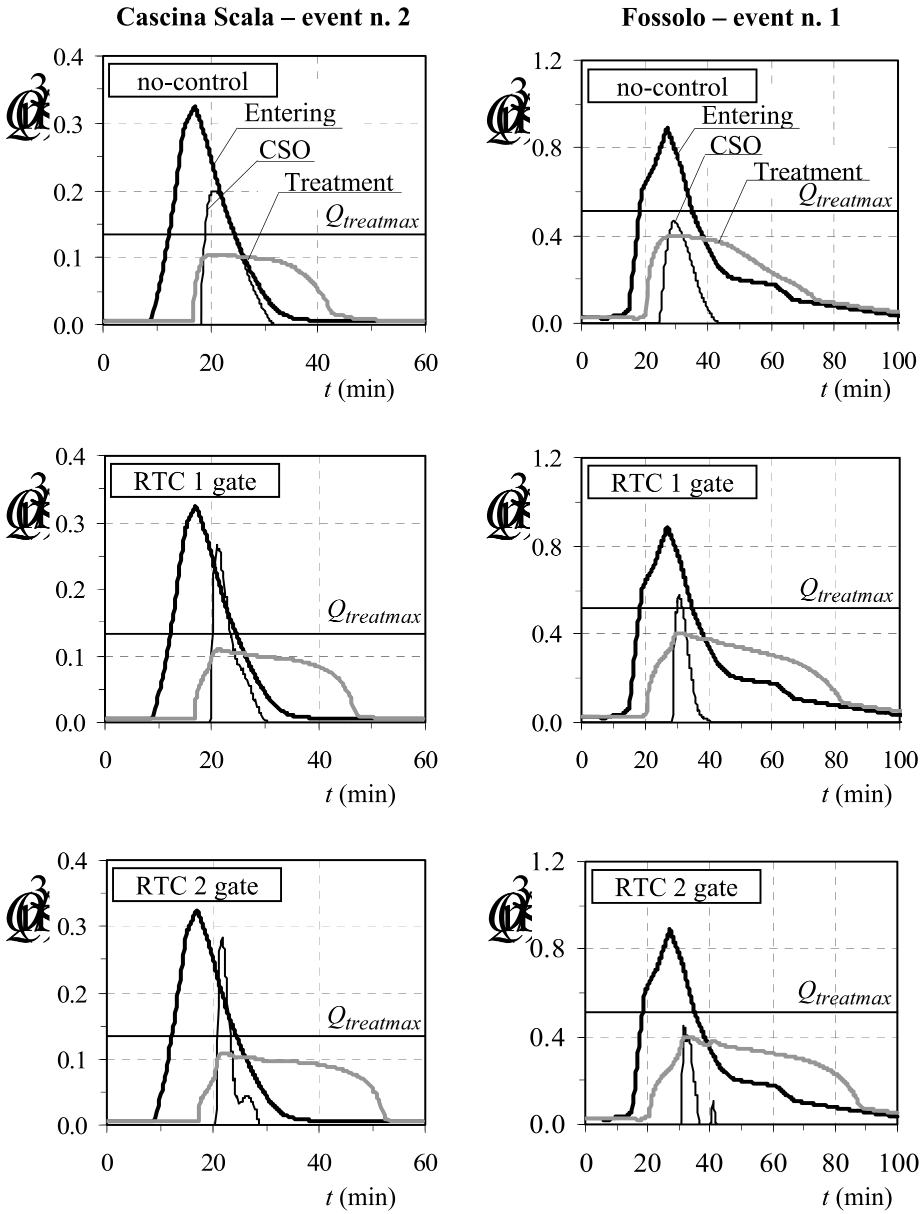

Figure 3. Entering flow rates, CSO discharges and flow rates conveyed to the treatment plant for the different scenarios and for the two catchments ($r = 6$).

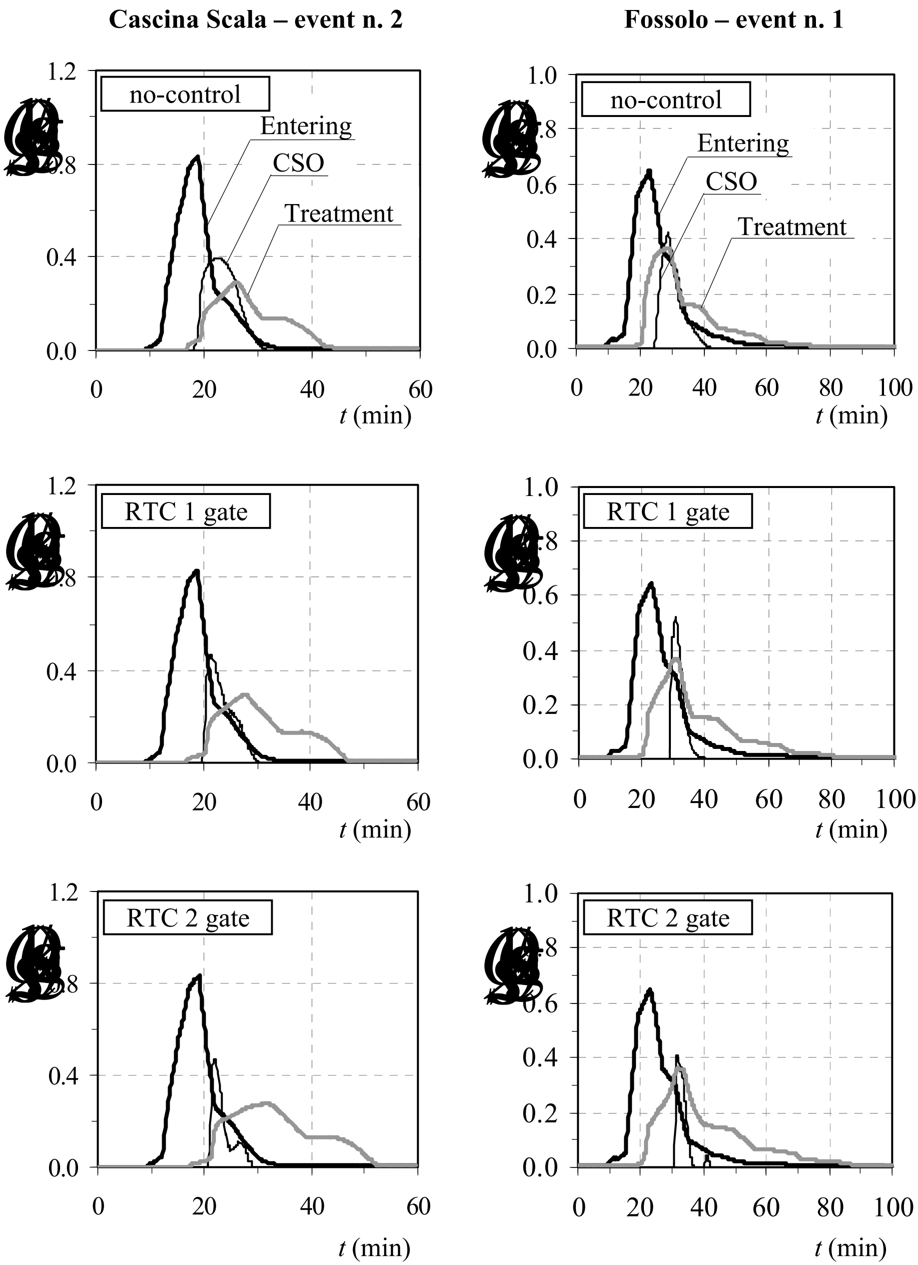

Figure 4. SS mass rates entering, discharged and conveyed to the treatment plant for the different scenarios and for the two catchments ($r = 6$).

Results of simulations for all the events for both urban catchments are summarised in Figures 5 and 6. In particular Figure 5 shows the ratio W between the total discharged water volume and the total inflow volume for the three examined scenarios and for two values of r. Similarly, in Figure 6 the ratio P between the total discharged SS mass and the total inflow SS mass is reported.

Globally, the figures show the improvement of the performances of the CSO device with the introduction of the RTC techniques. Significant average reductions (in comparison to the no-control scenario) of both total discharged water volumes (Table 5) and total discharged SS masses (Table 6) have been obtained for the two catchments.

Consequent average increases (in comparison to the no-control scenario) of both total treated water volumes and total treated SS mass have also been summarised, for the two catchments, respectively, in Table 7 and 8.

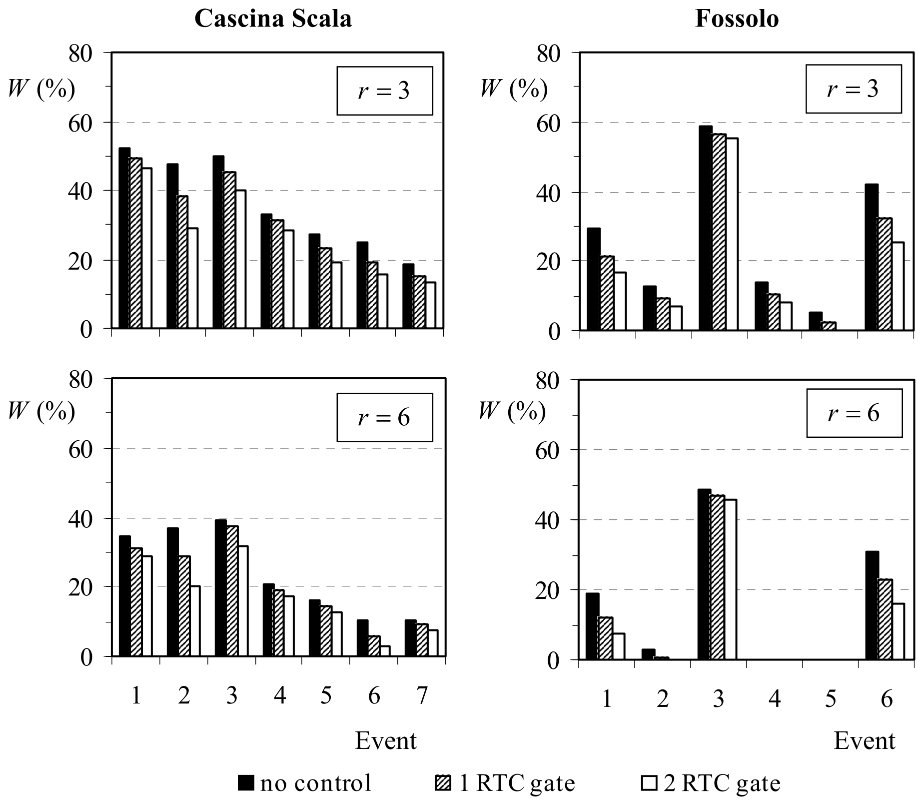

Figure 5. Ratio W between the discharged water volume and the total inflow water volume for various events, for three scenarios and two values of r.

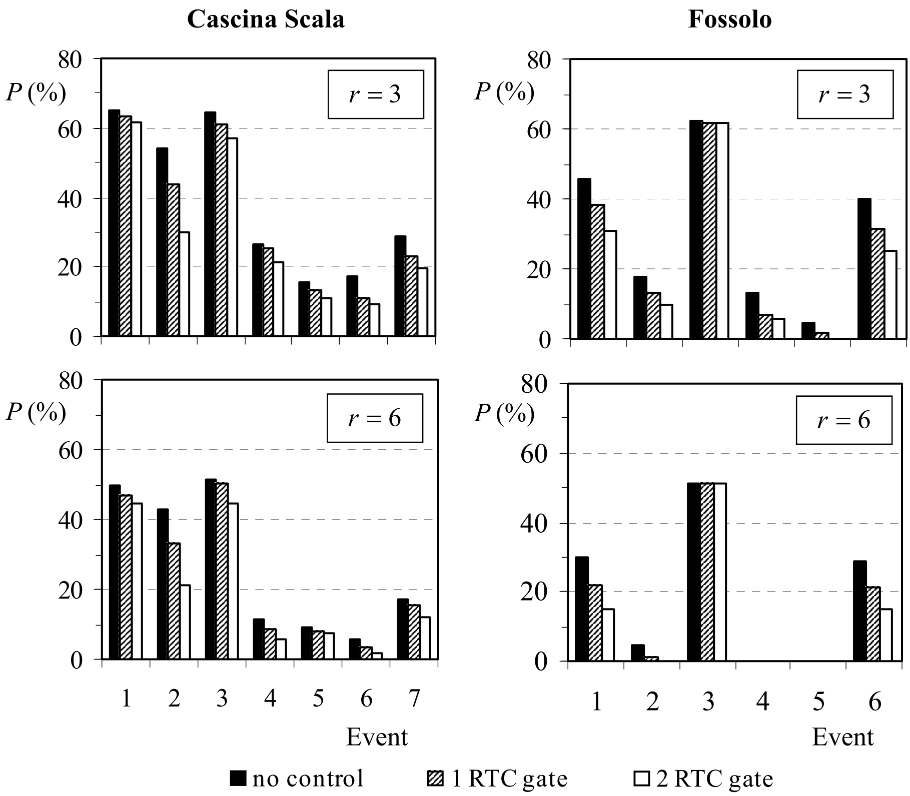

Figure 6. Ratio P between the discharged SS mass and the total inflow SS mass for various events, three scenarios and two values of r.

Table 5. Average reduction (%) of total discharged water volumes in comparison to the no-control scenario.

	Cascina Scala		Fossolo	
	$r = 3$	$r = 6$	$r = 3$	$r = 6$
one RTC gate	13.5	16.2	27.9	49.1
two RTC gates	25.5	31.0	46.4	63.1

Table 6. Average reduction (%) of total discharged SS masses in comparison to the no-control scenario.

	Cascina Scala		Fossolo	
	$r = 3$	$r = 6$	$r = 3$	$r = 6$
one RTC gate	14.5	17.1	28.6	46.4
two RTC gates	26.9	34.5	45.4	59.8

Table 7. Average increase (%) of total treated water volumes in comparison to the no-control scenario.

	Cascina Scala		Fossolo	
	$r = 3$	$r = 6$	$r = 3$	$r = 6$
one RTC gate	7.4	4.6	7.4	5.2
two RTC gates	14.7	9.6	12.3	8.9

Table 8. Average increase (%) of total treated SS masses in comparison to the no-control scenario.

	Cascina Scala		Fossolo	
	$r = 3$	$r = 6$	$r = 3$	$r = 6$
One RTC gate	7.9	4.9	7.4	5.2
Two RTC gates	16.8	11.6	12.6	9.3

6. Conclusions

Real time control (RTC) of sewer systems has been proven to be a cost-effective solution for reducing the qualitative impacts of both combined sewer overflow discharges into the receiving water bodies and shock loads to the treatment plant. In this paper the improvement of the performance of a quality-oriented CSO device by the application of RTC techniques to moveable gates is evaluated in terms of reduction of discharged volumes and pollutants. The potential benefits of RTC in terms of increase of water volumes and pollutant masses conveyed to the treatment plant have also been investigated. For these objectives a local control strategy based on the activation of the maximum in-line storage capacity has been adopted. The analysis has been carried out using a numerical model specifically developed for RTC applications. The implemented model is based on the fully-dynamic St. Venant equations for the flow and on the advection-dispersion equation for the analysis of pollutants in the sewer system. Simulations have been run with the quantitative and qualitative experimental data measured, during several rain events, in two Italian urban catchments, Cascina Scala (Pavia) and Fossolo (Bologna).

The benefits of the introduction of the real time control have been evaluated separately by comparing the results of RTC scenarios (with one and two controlled gates) with the results obtained in the no-control scenario. Globally, in spite of the good performance of the adopted CSO device, the results of the simulations show remarkable advantages in terms of reduction of CSO discharges obtained with the RTC of one moveable sluice gate and the additional benefits provided by the introduction of the second gate. An appreciable increase of the total volume conveyed to the treatment plant is also obtained without exceeding the maximum treatment capacity.

7. Acknowledgment

The experimental data for the urban catchments of Cascina Scala and Fossolo were provided by Prof. S. Papiri and Prof. C. Ciaponi from the University of Pavia and by Prof. S. Artina and Dr. M. Maglionico from the University of Bologna, respectively. Their help and advice are gratefully acknowledged.

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COMBINED SEWER OVERFLOWS INTO THE CRATI RIVER (COSENZA, ITALY) AND RETENTION STORAGE SIZING

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1. Introduction

It is well known that combined sewers are commonly used in urban drainage and that combined sewer overflows (CSOs) represent an environmental risk to receiving bodies [25]. In dry weather, control of the network operation is possible, whilst in wet weather combined sewer overflow structures represent safety valves of the sewer system, and for this reason they represent a preferred path for excess flows causing pollution of receiving water pollution [35].

Ashley et al. [3, 4] and Blanksby [7] report data about the extent of CSO structures in Denmark and U.K. Diaz-Fierros et al. [15] report severe effects of CSOs on the River Sar, in Spain.

The impacts of CSOs are pointed out, among others, by the ASCE – WEF Manual of Engineering Practice [2] listing the following observed pollutant concentrations (Table 1).

Table 1: Observed pollutant concentrations (after [2])

	TSS (mg/l)	BOD (mg/l)	COD (mg/l)
Range	273-551	59-222	264-481
Average	370	115	367

In Italy, experimental studies on water quality in urban catchments were recently conducted by universities and research centres, with the aim of assessing the pollutant loads in stormwater and developing a better understanding of the mechanism of pollution of receiving waters by CSOs [1, 14, 24, 26, 27].

The experimental data available until now show that combined sewer pollutant concentrations can be far above those allowed for discharge into surface waters; indeed there are many countries in which CSOs are the object of specific regulations, aimed to reduce the pollution of receiving water bodies. Roesner and Rowney [31] point out that during the mid 1970s and early 1980s it became increasingly apparent that pollution in wet weather discharges was significant and national regulations started to be introduced. The same authors point out that in the USA, the National CSO Control Policy was finalised in 1994, based on the principles of accurate

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characterisation of the combined sewer systems, demonstration of implementation of a number of minimum controls and the development and implementation of a long-term CSO control plan.

The EU Council Directive of 21 May 1991 concerning urban wastewater treatment (91/271/EEC) contains the following definitions:

'*domestic wastewater*': wastewater from residential settlements and services which originates predominantly from the human metabolism and from household activities;

'*industrial wastewater*': any wastewater which is discharged from premises used for carrying on any trade or industry, other than domestic wastewater and run-off rain water;

'*urban wastewater*': domestic wastewater or the mixture of domestic wastewater with industrial wastewater and/or run-off rain water.

Secondary treatment is standard for urban wastewater disposal into surface waters, and, consequently, CSOs are allowed only during exceptional events.

Nevertheless, in Europe most countries have a significant number of CSO structures and measures should be identified, based on the criteria listed below, to limit the pollution discharged in CSOs into receiving waters [32]:

- the dilution ratio between the sewer flow and the receiving stream flow;
- the capacity of the downstream sewer in relation to the dry weather flow;
- a specified number of overflows per year.

Which events should be considered exceptional, and what loads should be discharged during them, is a problem that can be solved only by taking into consideration the receiving water characteristics. The present paper aims to contribute to this discussion by analysing CSO volumes and loads observed in an experimental catchment in the City of Cosenza, in Southern Italy.

2. CSO mitigation

CSOs reduction is a necessity and the means for reaching such a goal vary. The approaches to reducing CSO loads in the USA consist traditionally in on-line or off-line retention storage [2]. Also in Japan the main solution consists in stormwater storage and treatment in wastewater plants [18]. In Germany, CSDT (Combined Sewer Detention Tanks) are the most common way of dealing with combined sewers in wet weather and storage facilities are considered an essential part of the sewer system [19]. In France many catchments are served by stormwater reservoirs, as reported by Faure et al. [17] for the City of Nancy, by Perez-Sauvagnat et al. [28] for Seine St. Denis, and by Charry and Lussagnet [13] for the City of Marseille.

It is well known that sediments carry much of the pollution loads (30 to 50% according to several authors, cited by Ashley et al. [4]). Many aspects of the mechanics of suspended transport and that of near-bed transport are still under study. In particular the weak resistance to shear causes rapid re-entrainment of deposited solids with the increase of discharge. This fact is especially evident for combined sewers, and influences greatly the impact of CSOs on receiving waters.

Retention capacities are the most used device to reduce pollutant loads; nevertheless their behaviour is known mostly empirically.

Recently, mechanics of sedimentation processes in ponds was taken into consideration [22, 23]. Other facilities can be used for reduction of CSO pollutant loads, like vortex separators [34], screens, special overflows structures [5], and high side weirs [21], either with or without storage.

The impact on treatment plants is a complex problem, which was addressed among others by Dormoy et al. [16]; they concluded that common plants need special considerations and install CSO tanks to treat combined sewer wastewater. In addition to retention storage, other solutions, such as detention storage and real-time control techniques were also recently proposed.

Detention storage allows detaining a part of the runoff volume in facilities built in the catchment and it is frequently associated with local runoff control. These possibilities are interesting, but the feasibility of their operation under full control is questioned [33]. Real-time control [12] allows the best use of in-sewer capacities, by using mobile devices and controls. Even though this technique is promising, for the time being, there relatively few practical examples of such schemes. It is very likely that in the years to come, off-line and on-line retention storage will remain the most practical way of controlling CSOs.

3. The urban catchment

The urban catchment of the Liguori creek, in the City of Cosenza, has been studied over many years by the University of Calabria. The area of the catchment is 413.62 ha in total, consisting of buildings (10.2%), gardens (0.8%), pavements and roads (37.6%) and an open natural area (51.4%). Originally, the main drain was a natural stream; today, because of the increasing urbanisation, its urban reach has become a combined concrete sewer with a horseshoe cross-section. Upstream of its outfall into the Crati River, a sewer collector intercepts the wastewater and conveys it to the treatment plant.

Three experimental stations for data collection are used in the catchment; the one considered in the present work is located at the catchment outlet (station 1) and includes a rain gauge, an ultrasonic water level gauge and an automatic sampler. Further information concerning measurement stations and the catchment can be found elsewhere [8, 9, 30]. The map of the catchment is shown in Figure 1.

The overflow structure was studied by means of a physical model, which was designed according to the Froude similarity [10]. When the overflow operates, the flow diverted to the treatment plant is almost constant and equals 650 l/s, which is about 3-4 times the dry-weather flow. Over 120 rainfall-runoff events were observed at this site [29].

For the purpose of sizing a retention storage tank, 23 significant events observed from January 1995 to January 2003 were selected, and the relative overflows were computed. Table 2 shows the rainfall-runoff characteristics and the overflows during these events.

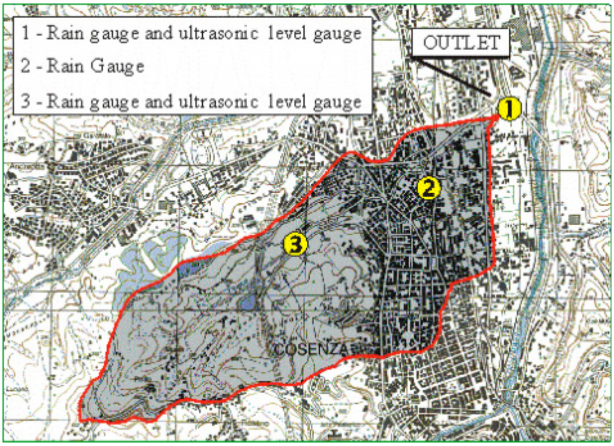

Fig. 1 – Experimental Catchment

Table 2. 23 events observed from Jan. 1995 to Jan 2003; *h* is the total rainfall, V_T and V_{OF} are, respectively, the total runoff volume and the overflow volume, t_s is the antecedent dry weather period, *Q* is the max discharge

Event	Date	Duration (min)	t_s (d)	h (mm)	Q (m^3/s)	V_T (m^3)	V_{OF} (m^3)	V_{OF}/V_T (%)
1	3/1/95	455	1	34,00	3,20	17738	4850	27
2	7/1/95	240	3	8,40	2,09	33557	14346	43
3	12/1/95	89	4	8,32	1,69	8757	2073	24
4	22/1/95	49	9	6,40	1,33	6055	1277	21
5	31/1/95	450	8	28,00	2,20	4501	844	19
6	14/2/95	310	3	24,40	2,20	30676	9022	29
7	16/2/95	165	1	15,20	6,08	12511	3157	25
8	18/2/95	580	1	28,00	2,80	10197	4762	47
9	25/2/95	460	5	42,80	6,52	24117	5036	21
10	2/3/95	1055	3	96,40	4,47	43009	21587	50
11	9/3/95	410	3	37,60	7,73	98968	47478	48
12	28/3/95	254	12	31,52	4,65	38023	18631	49
13	9/4/95	50	9	10,80	5,04	54672	13323	24
14	14/5/95	130	10	11,20	1,79	18681	6117	33
15	1/8/95	173	69	14,80	4,65	9276	2491	27
16	16/8/95	43	1	21,60	8,24	12132	5483	45

continues

Table 2 continued

Event	Date	Duration (min)	t_s (d)	h (mm)	Q (m^3/s)	V_T (m^3)	V_{OF} (m^3)	V_{OF}/V_T (%)
17	24/8/95	29	3	10,00	2,80	11300	7869	70
18	30/4/96	547	13	48,00	5,86	59623	28691	48
19	11/5/96	276	9	27,60	9,60	41435	16773	40
20	13/ 4/98	31	0,4	3,60	1,17	1919	182	9
21	1/11/00	110	4	12,40	4,84	6472	3263	50
22	24/2/02	411	2	18,00	1,51	10599	1496	14
23	29/1/03	179	3	19,20	5,04	24386	16210	66

4. Combined sewage quality

Dry weather discharges ranged from 150 to 250 l/s. The concentrations of BOD_5, COD, and TSS observed during dry weather are listed in Table 3 [8].

Table 3. Dry weather flow characteristics

	BOD_5 (mg/l)	COD (mg/l)	TSS (mg/l)
Range	12,5 – 185	21 – 392	1,3 – 279
Mean Value	87	170	87

Concerning the quality of stormwater, the paper considered seven events; for each of them the following parameters were observed: BOD_5, COD, total suspended solids (TSS), pH and conductivity [29]. For these events, the runoff volume, and the range and average values of pollutants studied are shown in Table 4.

Table 4. Wet weather flow characteristics

		range (mg/l)			average (mg/l)		
Event	V_T (m^3)	TSS	COD	BOD_5	TSS	COD	BOD_5
24 MAR 98	11.222	27-344	144-157	15-250	165	150	66.5
13 APR 98	1.919	224-296	273-354	150-160	250.7	303.7	155
28 APR 98	11.568	73-342	42-760	-	197.5	452.6	-
1 NOV 00	6.472	141-1333	160-1387	152-544	574	583.03	578.5
24 FEB 02	10.599	32-620	46-684	4-201	194.76	229.43	79.54
23 MAR 02	2.673	116-193	212-310	95-125	158.6	267.4	116
29 JAN 03	11.222	83-4487	54-311	-	1335	96	-

In total, 112 samples were analysed. A regression analysis was carried out for BOD_5, COD and TSS, and the results are listed in Table 5 [11].

Table 5. Regression analysis for BOD and COD (after [11])

Regression Equations	R^2
COD = 0.76 TSS+143.77	0.79
BOD_5 = 0.36 TSS+27.87	0.83

The event of 29 Jan 2003 was not included in the regression analysis because of unusual suspended solids loads caused by road work.

The pollutographs showed that pollutant concentrations have the same general pattern as sewer flow, Q, and that concentration peaks correspond generally to discharge peaks of each rainfall event [11].

5. Pollutant mass evaluation

For the evaluation of pollutant loads, four of the events in Tab. 3 were considered: 13 April 1998, 01 Nov 2000, 24 Feb 200 and 29 Jan 2003.

Solids build-up during dry periods was simulated by an exponential equation:

$$Ma = \left(\frac{Accu}{Disp}\right) \cdot A_i \cdot \left(1 - e^{-\left(\frac{Disp \cdot t_s}{24}\right)}\right) + M_r \cdot e^{-\left(\frac{Disp \cdot t_s}{24}\right)} \tag{1}$$

where Ma is the total mass accumulated on the catchment surface [kg], $Accu$ is the build-up rate [kg/(ha·day)], t_s is the time elapsed from the previous rainfall event [h], $Disp$ is the decay coefficient [day^{-1}], M_r is the residual mass at the end of the event immediately before the one being considered [kg], and A_i is the impervious catchment area [ha].

In the present paper, the lack of data for continuous simulation made it necessary to assume M_r= 0; moreover, since the events represent high intensities and for all of them overflows occurred, we assumed that the mass accumulated was completely washed-off during the event, that is $Ma = Mw$. As shown in Table 6, the parameter values $Accu$ = 6 kg/(ha·day) and $Disp$ = 0.08 day^{-1} provided good simulations of the TSS loads. Besides the total accumulated mass, Ma, Table 6 also includes the values of the total mass observed, M_{OBS} .

Table 6. Observed and calculated masses

Event	M_{OBS} (Kg)	Ma = Mw (Kg)	Event	M_{OBS} (Kg)	Ma = Mw (Kg)
13 APR 98	260	503	24 FEB 02	1808	2360
1 NOV 00	3930	4371	29 JAN 03	3124	3405

It is more difficult to evaluate correctly the pollutographs for individual events, as done in other works [9, 10, 11], because errors in peak concentrations and time of their occurrence appear to be significant. Even though a variability of concentrations during events exists, only event mean concentrations were considered in the following analysis.

6. Retention storage capacity

According to the EU Directive, uncontrolled discharge of combined overflows into receiving waters must be avoided, so it is necessary to provide a storage capacity and an adequate treatment. This clearly demands considerable retention capacities. According to Paoletti et al. [26] for the Italian climate storage capacities up to 200 m^3/ha are needed.

For the case under consideration, we must consider that a considerable amount of the runoff volume V_T is diverted to the treatment plant. For the 23 events in Tab. 2, one can see that the ratio V_{OF}/V_T ranges from 9 to 70%, with an average of 36%. This often allows a considerable reduction of overflow volumes.

For the 23 events in Tab. 2 the mass of pollutants accumulated, *Ma*, was computed by eq. (1), then the mass washed off, *Mw*, was assumed equal to Ma, the average concentration *c* was evaluated as the ratio Mw/V_T, and the mass discharged by the overflow structure, $M_{OF,}$ was consequently calculated as the product of $c \bullet V_{OF}$. The volume diverted to the treatment plant, V_{TP}, was evaluated as the difference V_T - V_{OF}. The computed values are listed in Table 6.

Table. 6. Volumes and pollutant masses: total and overflow values

Event	V_T (m^3)	V_{OF} (m^3)	V_{TP}	Mw (Kg)	c (mg/l)	M_{OF} (Kg)
1	17738	4850	12888	1227	69	335
2	33557	14346	19211	3405	101	1456
3	8757	2073	6684	4371	499	1035
4	6055	1277	4778	8191	1353	1728
5	4501	844	3657	7544	1676	1414
6	30676	9022	21654	3405	111	1002
7	12511	3157	9354	1227	98	310
8	10197	4762	5435	1227	120	573
9	24117	5036	19081	5262	218	1099
10	43009	21587	21423	3405	79	1709
11	98968	47478	51489	3405	34	1634
12	38023	18631	19392	9849	259	4826
13	54672	13323	41349	8191	150	1996
14	18681	6117	12564	8789	470	2878
15	9276	2491	6786	15896	1714	4268
16	12132	5483	6648	1227	101	555
17	11300	7869	3431	3405	301	2371

continues

Table 6 continued

Event	V_T (m^3)	V_{OF} (m^3)	V_{TP}	Mw (Kg)	c (mg/l)	M_{OF} (Kg)
18	59623	28691	30932	10319	173	4965
19	41435	16773	24661	8191	198	3316
20	1919	182	1737	503	262	48
21	6472	3263	3209	4371	675	2204
22	10599	1496	9103	2360	223	333
23	24386	16210	8176	3405	140	2264

As shown in Fig. 2, the max values of overflow volumes correspond to the lowest concentrations.

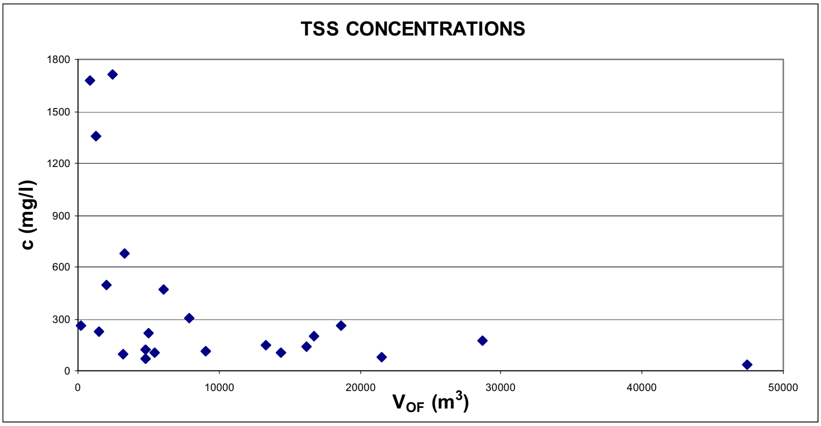

Fig. 2: TSS concentrations versus CSO volumes

In any case, the pollutant concentrations in CSOs are significant and in most cases higher than the concentrations observed in wastewater (90 mg/l). The volumes diverted to the wastewater treatment plant do not exceed 3 times the daily dry-weather flow volume.

If we assume an off-line retention tank of capacity V_R, the overflow volume V_{OF} is transported into it; once this capacity is exceeded, the volume $V_D = V_{OF} - V_R$ is spilled into the receiving body. The masses M_D discharged as the result of this overflow were computed taking into account the average concentrations for each event. Five different retention capacities were considered, from 10,000 up to 50,000 m^3, that is from 50 to 250 m^3/ha.

It is known that there are different ways of assessing the performance of retention facilities [6, 20]. In this paper, only the following indicators were considered (Tab. 7):

- Maximum values of discharged volume $V_{D,MAX}$,
- Maximum values of mass discharged into the river $M_{D,MAX}$
- Maximum concentrations spilled to the river c_{MAX} , and
- Number of overflow events, n_e .

Table 7. Retention tank performance indicators

V_R (m^3/ha)	50	100	150	200	250
$V_{D,MAX}$ (m^3)	37478	27478	17478	7478	0
$M_{D,MAX}$ (Kg)	3235	1504	601	257	0
c_{MAX} (mg/l)	259	173	34	34	0
n_e	8	3	1	1	0

One can observe that the maximum values of discharged volumes, V_D, do not correspond to the same events producing maximum values of discharged masses, $M_{D,}$ and concentrations *c*.

We can conclude that, while a retention capacity of 50 m^3/ha looks inadequate, a capacity of 100-150 m^3/ha could reduce the number of spills to the receiving waters to 3-1, provided that a discharge of 3-4 times the dry-weather flow is continuously diverted during storms to the treatment plant.

A retention capacity of 100 m^3/ha could reduce the spill events to 3, with a maximum discharged mass of 1504 kg, that is about the value treated daily by the wastewater plant during dry weather. A retention capacity of 150 m^3/ha could reduce the number of spills to 1 and the mass discharged during this event would be equal to 1/3 - 1/2 of the wastewater volume daily treated by the wastewater plant during dry weather, and with a relatively low concentration. The values of discharged masses and concentrations, together with considerations of flow and ambient water quality in the river, should suggest the correct choice of the storage size.

7. Conclusions

The paper aims to contribute to the CSO pollution mitigation by retention storage, taking into account the observations carried out in an experimental catchment in Cosenza, Italy. A few observed events allowed simple simulation of pollutant build-up and wash-off, serving to obtain average wet-weather flow pollutant concentrations. Consequently, for 23 observed events, volumes, masses and pollutant concentrations in overflows were computed. Assuming five different sizes of retention storage, from 50 to 250 m^3/ha, volumes, masses and concentrations of final overflows into the receiving water body were evaluated. A retention capacity in the order of 100-150 m^3/ha reduced overflows to 1 to 3 events per the time period studied. The values of pollutant masses and concentrations in overflows, in relation to the flow and ambient water quality in the receiving water (river), should serve to choose the correct storage size.

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MODERNISATION PROBLEMS OF THE SEWERAGE INFRASTRUCTURE IN A LARGE POLISH CITY

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1. Introduction

The combined sewerage system has been implemented in many European cities at the end of the 19th and in the beginning of the 20th century. It seemed to be advantageous to drain all kinds of sewage out of the city centre. In addition, urban rivers were simultaneously channelised as "unnecessary" watercourses inside the urban environment and were incorporated into the sewerage infrastructure as transport elements for disposal of stormwater and overflow wastewater.

In the course of time, residential, commercial and industrial areas continued to develop within the city boundaries. Non-sealed (pervious) surfaces gradually disappeared, and the sealed impervious surfaces became larger and larger and smoother. As a consequence, the basic combined sewerage system revealed its shortcomings. Urban floods appeared more frequently, combined storm overflows also operate too frequently and wastewater treatment plants are overloaded during wet weather periods. At present, the City of Lodz, situated in central Poland, is a typical example of such problems.

2. History of the city sewerage development

The City of Lodz was a rapidly developing industrial centre starting in about 1850. The dominant industry was water consuming textile industry developed by local expansive manufacturers. This required an effective sewerage system. Due to financial problems, only the invited British engineer William Heerlein Lindley designed the sewerage system in 1909 [1]. Engineers from the Lindley family were also known as designers of sewerage in such cities like Berlin, Hamburg, Budapest and Warsaw. The combined system has been chosen for Lodz despite a strong substantiation of advantages of the separate sewer system! The main reasons for this decision were the possibility of avoiding the construction of the required stormwater treatment facilities and using stormwater as flushing water for a combined system. Another important reason was the expectation of lower construction and maintenance costs of the combined sewer network. W.H. Lindley finally decided to use the

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natural relief of the terrain and the local hydrological situation for designing the main trunk sewers along the natural watercourses. So those trunk sewers have been situated in several river valleys, which made it possible to use small urban rivers as receiving waters for storm overflows. Lindley's Latin numbering of trunk sewers (I, II, III and IV) survived till today and was extended (up to VIII) for sanitary trunks sewers only.

In total, 6335 hectares of the city area were foreseen to be serviced by the combined sewerage system. The combined sewer overflow (CSOs) structures were designed for the initial dilution ratio equal to 6, on the average, and to 4 for the maximum dry weather flow.

The combined system containing 21 CSOs (not all the overflows proposed by Lindley were eventually constructed) was implemented for the central district of the city in the 1930s as a part of public works program. The construction continued after 1945, approximately until the late 1960s. However, after 1945 the spatial expansion of the combined system (i.e., expansion outside the Lindley's drainage area limits) turned out to be impossible, because of difficulties in extending sewer lines with a proper slope. Therefore, a separate sewer system has been implemented for all new city districts outside the centre. But the dry weather flow had to be connected to the existing combined system, thus creating serious problems to many CSO structures and the receiving waters. Also, some the so-called relief sewers were added to the combined system to protect some "sensitive" urban areas against flooding. These sewers receive input from other adjacent sewers through diversion manholes. Thus, these non-typical sewers function just as specific additional CSOs.

Although Lindley was unable to use a credible Intensity-Duration-Frequency (IDF) formula for design rainfalls, he had done a lot of work collecting rainfall data from various meteorological stations located in present Poland, Russia and Germany. After applying a regression analysis to data from 116 stations, he obtained a formula that luckily appeared close to the contemporary one (Table 1). Lindley, however, was not able to take the return period as a variable into consideration.

Table 1. Comparison of the Lindley's and contemporary formulas for intensity of design storms

Lindley's formula (1909)	W.Błaszczyk's formula (1967)
$i = \dfrac{225}{t^{0.45}}$ [mm/h]	$i = \dfrac{169\sqrt[3]{T}}{t^{0.67}}$ [mm/h]
Data from 58 meteorological stations and 116 intense storms measured during the period of 1891-1907	Data from the Warsaw station obtained during the period of 1837-1959

T –return period [years], t- storm duration [min]
Both formulas are valid for the average intensity, equal to the whole duration

Also, he was very careful in the choice of the runoff coefficient that was assumed to be of 0.70 for the city centre and of 0.45 for the suburbs. Next, hydrological calculations were performed with the rational formula, nowadays known as the Buerkli-Ziegler formula with the reduction-delay coefficient φ expressed as a function of the catchment area. Lindley also proposed the trunk sewer cross-sections, which survived until today for sewers, which were mostly built of brick.

The combined sewerage system has then been maintained almost without any changes in its layout during the next 40-70 years after its construction. The system appeared to work well for both dry and wet weather flows. Pollution of the receiving waters was observed for a long time, but it was caused by untreated dry weather flow before construction of the Lodz wastewater treatment plant (WWTP). So, the wet weather pollution effects were masked or even viewed as a positive dilution of the polluted river water. Furthermore, the problem was not assigned high priority, since most of urban rivers had been covered. In the case of Lodz, the pollution problem has been "shifted" to the downstream reaches of the main receiving water and observed at long distances outside of the city itself. Thus, the situation was not stimulating for looking for a corrective action. A great volume of dry weather flow, i.e., a mix of raw domestic and industrial sewage was still flowing out of the city area during dry weather periods, and also during wet weather periods. Paradoxically, the situation has changed towards better in the 1990s, because of the collapse of the textile industry. Thus, a gradual decrease of water consumption is still being observed.

In the 1990s, the city built the so-called Lodz Agglomeration Group Wastewater Treatment Plant (LAGWWTP), one of the biggest in Europe, receiving sewage from Lodz and several satellite towns with the present mean daily dry weather flow of about 170,000 m^3/d. This flow increases up to 300,000 m^3/d and more during wet weather.

3. The present situation and needs for modernisation

About 20-30 years ago, the situation seemed to be better and still no action seemed to be necessary. Therefore, new impervious surfaces in the central city districts were draining to the existing combined system without any detailed control of its capability. However, new state regulations introduced in November 2002 (valid from January 1st, 2003) again changed the situation dramatically. The regulations had been prepared before the expected Poland's entry into EU and the regulations harmonisation (at least partial) with the EU water laws was assumed. Since 2003, any urban combined system should be improved if required, to meet the required limit of 10 CSO spills per year with, a three-time initial dilution as a minimum.

Many sewer structures designed around 1910 are so complicated that it is almost impossible to reconstruct the reasons for their design. For instance, there are large chambers with connections to non-existing trunk sewers, intentional dry weather flow channels inside channelised riverbeds, overflows from one sewer to another without any apparent necessity, very long (up to 100m) overflow weirs, and so on (Fig.1). Anyway, such structures do not seem to work properly nowadays although they might have been useful for a system, which has not been completed. Furthermore, both dry and wet weather flows have changed in the course of time, as described above.

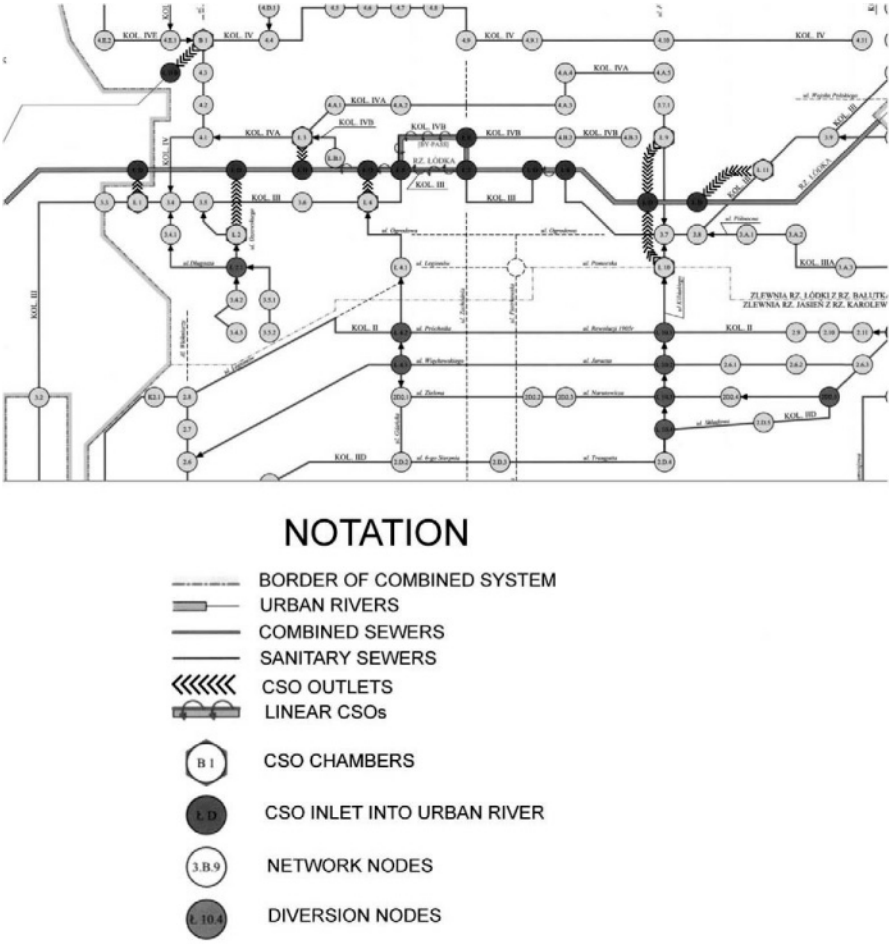
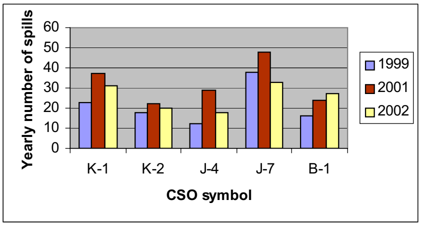


Fig.1. A part of the Lodz combined sewerage system in its present structure

In 1999 the Lodz combined sewerage was equipped with several automatic ultrasonic measurement stations at overflow structures. The detected annual number of spills turned out to be much greater than the allowed 10 (reaching 40 in a typical year). Moreover, the annual number of spills has a positive trend that clearly can be connected with the extension of the impervious urban surface connected to the combined system (Fig.2). Also, both dry and wet weather flows have changed in the course of time. Despite the decreasing water consumption, in most cases, the initial dilution of dry weather flow, i.e., the dilution when the spill starts is now about 2-2.5 only, and this is a very unusual result when considering the earlier hydraulic assumptions. Perhaps this could be explained by the simplifications made in the former Lindley's calculation procedure.



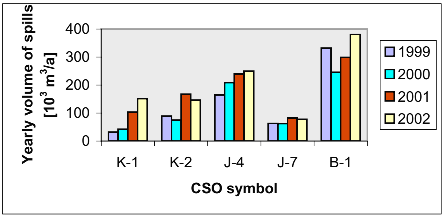


Fig.2. Annual number and volume of CSO spills in the Lodz combined system

Recently, some intense storms caused serious floods and damages. In practice, one can expect one severe urban flooding per year. Without considerations of climate change, one should assert that these storms were uncommon and greater than those statistically determined for their duration (Fig. 3).

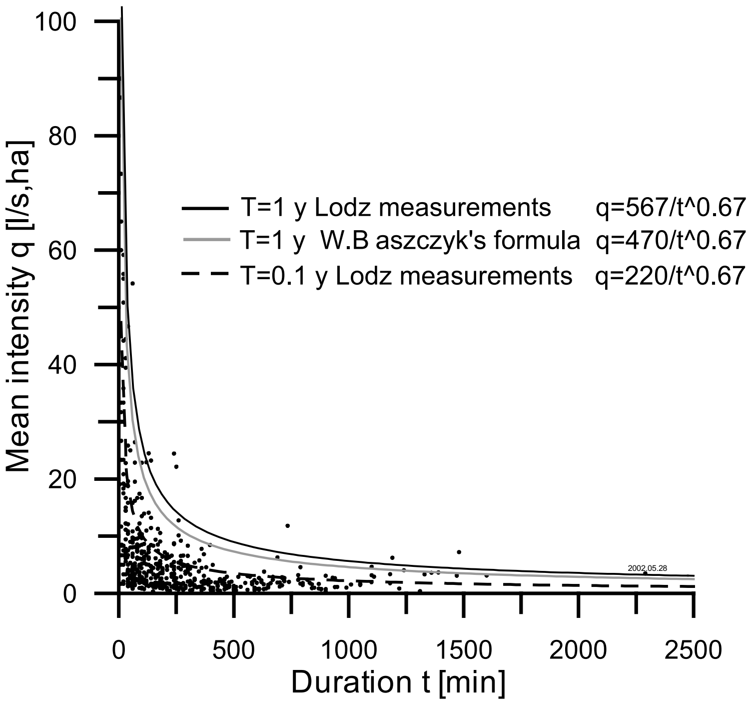


Fig.3. Rainfalls measured in Lodz from 1993 to 2002 and their IDF curves

Sometimes, more than one spill event can be observed during a day (Fig.4). Therefore, it was assumed that one day with spills means one spill event.

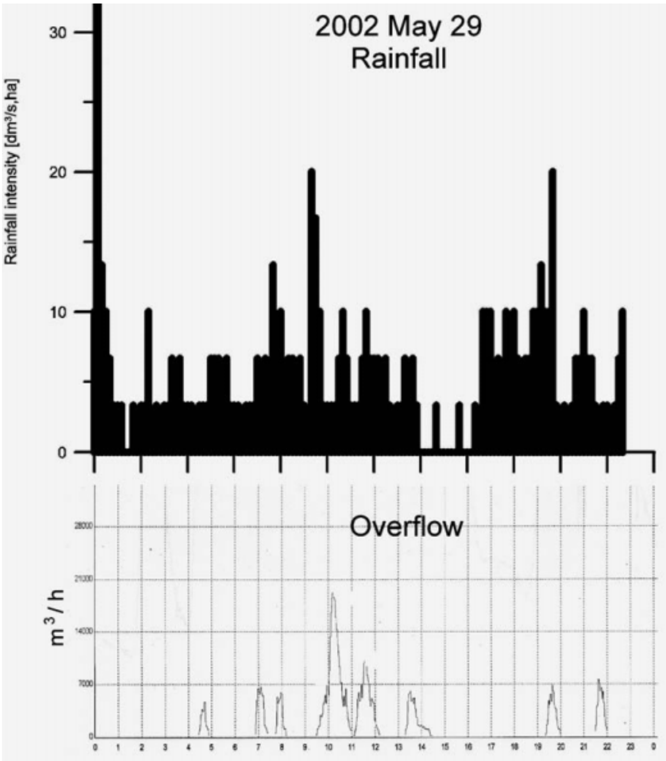


Fig.4. An example of rainfall-runoff measurements. Multiple spills according to the variable rainfall intensity are visible.

In 2002, a master plan for the combined sewerage system improvement was commissioned through open tender. Its main goals were:

- Reduction of combined sewer overflow frequency
- Reconstruction of the bottlenecks in the network to reduce the risk of urban floods
- Modernisation of the LAGWWTP, and
- Protection and improvement of the receiving water quality.

However, only the first goal was fully taken into consideration in the first stage of the master plan, which should be implemented by 2020. It is clear, however, that the other goals will have to be implemented soon as well.

3. The modernisation plan

First of all, the design work had to start with an inventory, which was performed by the design office MW Projekt Lodz, the winner of the competition, in autumn 2002.

During the last 40-70 years, the condition of the combined sewers was quite good, as the quality of construction works was high, especially in 1930s (by the way, much better than the quality of works performed 40-50 years later). However, many gaps and simple mistakes in technical documentation were discovered. Finally, some damage of trunk sewer structures was observed mainly due to corrosion of material, dynamic vehicle loads and unexplained "forces of nature". Brick material used for sewer construction appeared to be of excellent quality, however, corrosion of cement joints caused by industrial wastewater was observed. Illegal connections to the network were discovered, too.

After preliminary analyses, addition of detention storage at the CSO structures and elimination of unnecessary CSO chambers and segments were recognised as the most feasible options for upgrading the combined system. At present, there is no designed detention in the system at all. The plans for improvements are quite similar to those well described in the literature [2,3].

Also, the existing combined sewerage catchment has been analysed with respect to disconnecting some parts from combined sewers and directing wet weather flow into urban rivers. This turned out to be feasible only in some cases and resulted in reducing the total area of the combined sewer catchment by up to 17%. For individual CSOs, contributing area reductions as high as 30% were possible. Usually, the relatively easy way was to disconnect certain streets and the adjacent built-up strips of the catchment. This will be possible, however, only by constructing additional separate storm sewers, which is not easy in view of the existing urban infrastructure.

4. Design calculations and implementation problems

The measurement results are available for 7 out of 21 CSOs. The results were used for analysing measured overflow events and comparing them with a local rainfall database. A formula for critical rainfall characteristics was established, where the critical rainfall is defined as the one that activates CSOs. The whole 10-year rainfall database was then filtered in order to obtain rainfalls, which might be causing spills.

In general, the selection of critical storms was not simple for three reasons:

a) Statistical selection of the critical storm is easy - it is the rain that is exceeded by 10 larger storms during the year (see Fig.3). Nevertheless, it may be done only by neglecting intensity variations during the rainfall event, i.e., taking into consideration only duration and mean intensity.

b) Determining the critical rainfalls with the use of in-situ flow measurements at CSO structures gives better results, but such results depend on local characteristics of both the CSO and its adjacent catchment.

c) Variability of rainfall intensity plays an important role for CSO spills: average rainfall intensity itself is not reliable for proper modelling of CSOs.

Fortunately, the typical rainfall intensity for critical rains was found to be around 5-10 litres per second and hectare, and simultaneously the duration of such rainfall usually was not short. One can neglect the spatial distribution and assume uniform rainfall intensity over the whole catchment.

A combination of the "statistical" and "measured" critical rainfall was then used for the selection of all the suitable actual rainfalls. All the rainfalls with their characteristics close to theoretical critical one were selected as well as a number of rainfalls of somewhat smaller and greater intensities. Such a sub-database was then used for computer simulation of some variants of the upgraded combined system.

The HYDRA package (Pizer Inc. Seattle, U.S.A.) was used for this purpose. The software allows performing full dynamic modelling of the system including detention facilities and overflows. The software also allows for easy input of any symmetric sewer cross-section, which was very helpful for the master plan. The digital city maps were used for the input of the catchment area and the coefficient of imperviousness. This coefficient, by the way, was found to be greater than that of Lindley for the city centre (as it can reach 0.83) and smaller for suburbs and modern residential areas (0.25-0.35). The software was then calibrated with the use of measurements of flows and rainfalls (using 3 electronic rain gauges SEBA RG50 situated in the reference catchment of about 400 hectares).

The required detention tanks were sited on a preliminary basis, always upstream of each CSO. The tanks were designed being fed from the inlet trunk sewer via an additional auxiliary overflow protecting the existing one (Fig.5). The protected overflow node may only be activated if the tank is overfilled. Depending on space available, the tank may also be fed from the existing overflow sewer. In both cases, partial gravity emptying of the tank back to the trunk sewer seems to be possible and a small pumping station has to be used for complete emptying. The emptying time should not be greater than 24 hours, however, this parameter may be easily controlled by the pump capacity (in general, it is one of the known RTC procedures).

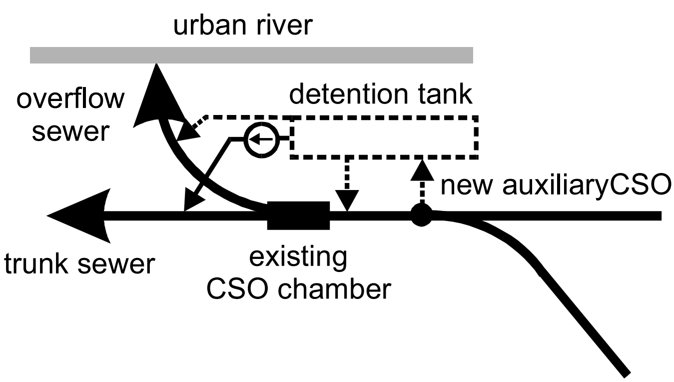


Fig.5. Scheme of possible modernisation of a single CSO system

Furthermore, the reconstruction of some CSO chambers is needed. It includes rising the overflow weirs by 15 to 50 cm and reconstructing (for instance doubling the capacity of) the outlet trunk sewers. Rising overflow weirs without this reconstruction contributed little to diminishing the spill frequency.

Implementation of all the described solutions in the catchment and the network is necessary to fulfil the regulatory limits and to assure safe functioning of the sewerage system, which should be free of sewer surcharge events for 5-year design storms.

Unfortunately, there will be many challenges in the implementation of these ideas. In general, one can say that the present city layout is poorly suited for any upgrading of its sewerage and no simple solutions exit. Almost all of the overloaded big CSOs are situated beneath the city centre. The individual detention volumes required, which would guarantee the appropriate overflow frequency, range from 500 m^3 to as much as 10,000 m^3. The upper limit requires a large space in the urban underground infrastructure and this space has to be found in the near future. Non-typical tanks like super pipe reservoirs should be proposed. In summary, a relative detention volume index of 10 m^3/hectare can be recommended for upgrading the combined system. This is an order of magnitude smaller than recommended earlier for a radical reduction of stormwater flow [4].

Applications of other detention facilities like Stormcell type boxes or infiltration gravel ditches were analysed as well. Such solutions might be attractive alternatives. However, in this case non-technical problems may play an important role: such measures should be implemented on property that does not belong to the city, e.g., private grounds and properties of developing companies and co-operatives, which are typical for Polish urban areas. So, the decision has to be postponed to the future. Besides, there is no appropriate regulation limiting the stormwater inflow into combined systems. A drainage fee (not very high) was recently introduced for impervious surfaces but only when stormwater is drained directly into the receiving water. Moreover, each building (according to the regulations) must be provided with storm drainage connected to a street storm or combined sewer unless it is technically impossible.

The so-called “distributed drainage and detention” have been also proposed for roof drainage in all residential settlements of “block houses” and possibly for some road drainage as well.

For infiltration there is little space and, unfortunately, a quite complicated geological structure – postglacial layers of variable soil characteristics ranging from clay to sand. Moreover, Lodz is situated above a natural aquifer containing high quality groundwater, which is protected against possible pollution. However, it was assumed that for critical storms, slow infiltration, even if feasible, is not hydraulically significant in the first approximation.

The upgraded system was then checked for large storms and the resulting pressurised flow and local flooding. It was noted that after implementing the detention facilities, such problems practically disappeared.

Disconnecting of some areas from the combined system will cause additional hydraulic loads to urban rivers. The master plan for rivers is being developed just now and extensive detention in ponds situated in riverbeds is foreseen. However, one has to add that reducing the CSO spills will compensate this additional hydraulic loading. Moreover, this option makes it possible to restore some segments of urban rivers [5,6].

All those proposals are foreseen as a general direction and they have to be implemented gradually together with developing the monitoring of sewerage and rainfall. Any reasonable conclusions obtained from that monitoring should serve for correcting the upgrading plans.

5. Reducing pollution of receiving waters

In spite of the importance of the need to modernise the Lodz sewer system, the solution has to be implemented gradually. However, all the proposed solutions have been treated as those of secondary importance.

As stated in the earlier section, the pollution problem in the Lodz receiving water is not typical, because the city represents the source area of this problem. The mass of pollutants is transported away from the city and causes detrimental effects downstream of Lodz urban agglomeration. The pollution effects are observed in other, downstream administrative districts ("voivodeships"). Plans were made to construct some small detention reservoirs on the main Lodz receiving water body, the Ner River, which transports urban dry and weather flow from Lodz. Detention tanks situated on the sewer network would likely remove the problem of the first flush completely. Moreover, possible spills would be much more diluted than present ones.

The application of overflow treatment facilities has been studied on a preliminary basis. Unfortunately, there are no state regulations regarding the CSO spill quality. Therefore, the following important questions arose:

- Where should the treatment facilities be located, upstream or downstream of the detention tanks? (Also, the hydraulic capacity should be considered.)
- Should the detention tank be used as a sedimentation tank? If yes, what about the required treatment efficiency, tank cleansing and sludge management?
- In the case of Vortex type facilities, how to manage the sludge trapped in it? Can the facility be design to transport underflow with sludge automatically away and where should it be directed?

A preliminary location of the treatment facilities for overflow sewers was proposed. This option seems to be more favourable and easier from the point view of construction than inserting fixed or rotating screens inside the CSO chambers. Flushing the trapped mass of gross solids into the near combined sewer may then be possible. Such technologies, as those promoted by the Hydro International in the form of automatic hydro-cleansing mesh strainers seem to be suitable for this goal.

At the LAGWTTP itself, the proposed upgrading of the combined system, however, will cause the following effects:

1. Equalisation of the wet-weather flow so that it might be observable up to 1 day after the storm, and
2. Increasing the total wet-weather flow volume to be treated.

Therefore, adding stormwater detention tanks to the LAGWWTP cannot be avoided, even after the planned extension of the plant capacity from about 170,000 m^3/day at present to 250,000 m^3/day in 2005. Currently, this problem is being investigated intensively.

A preliminary estimate of the needed detention volume ranges between 100,000 m^3 and 400,000 m^3, depending on some technological assumptions. Application of chemical settling aids (coagulants) in 2003 improved the technological effect, but more experience is still needed. First results are encouraging. In spite of this, the biological treatment process seems to be quite sensitive to disturbances, but not only to those caused by the wet weather flow. Disturbances can sometimes be observed during dry weather and manifest themselves as poor efficiency of the nutrient removal. The winter period is also critical in plant operation. The treatment efficiency is reduced, because of the temperature drop and additional inflow of stormwater or snowmelt, which cause a very distinct deterioration of the effluent quality.

Adding detention facilities both in the sewer network and at the LAGWWTP will have another obvious advantage – the possibility of implementation of real-time control. This aspect has not been studied in detail until now, but it appears to be promising, especially for the LAGWWTP operation. It was found that the present dry weather loading of the treatment plant is quite variable during the course of the week showing distinct decrease in loads on the weekend and, of course, each early morning. The use of RTC could equalise the loading of treatment facilities and could directly optimise the utilisation of detention tanks volume.

6. Concluding Remarks

Modernisation of combined sewerage systems in contemporary cities turns out to be a very difficult task. The lack of space and possible obstructions of construction works require selection of non-typical upgrading solutions or adaptation of the standard ones. The City of Lodz, one of the largest cities in Poland, started with a modernisation plan for its old combined sewerage system with 21 CSOs connected to a new large WWTP. In order to fulfil new state regulations, several technical proposals have been developed: (a) disconnecting some urban areas from the combined system, (b) adding a certain number of underground detention tanks at the existing CSOs structures, (c) rising the CSO weirs, (d) implementing distributed detention in the form of Stormcell devices for controlling roof runoff, and (e) creating detention in urban river courses. In the near future, addition of satellite treatment facilities at CSO outlet sewers must be also considered. Despite of their long-term operation, trunk sewers appear to be structurally sound and not damaged or corroded; nevertheless, there is a need to perform rehabilitation works on some network segments. The proposed solutions will significantly reduce the risk of urban flooding and this is an additional advantage of the future modernisation works. Successful demonstrations of detention facilities and real-time control for the sewerage network and the treatment plant in the city seem to be beneficial.

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EXTENSION OF THE VIENNA MAIN SEWAGE TREATMENT PLANT

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1. Introduction

Vienna, the capital of Austria, is situated on both sides of the Danube River (mean discharge ~1700 m^3/s), and has an actual population of about 1.6 million inhabitants. The wastewater is collected in the city mostly by a combined sewer system, which is served by the Main Treatment Plant of Vienna (MTPV). The plant is designed for a pollution load of 2.9 million population equivalents (PEs). The actual plant went into operation in 1980 as a mechanical biological treatment plant. The legal requirement (permit) was to reach a BOD effluent concentration of 70 mg/l. The biological treatment therefore consists of a high-load activated sludge process (sludge age ~ 2 d). The actual pollution influent load is about 3.3 million population equivalents, i.e., about half of the pollution load is derived from trade and industry. The wastewater can be characterised as relatively concentrated typical municipal wastewater (BOD_5 > 300 mg/l). The sewage sludge production (~200 t DS/d) of the plant is incinerated on site and ashes go to landfill. The plant does neither meet the EU requirements for normal areas [1] nor the corresponding more stringent Austrian minimum requirements to be met by 2005 at the latest.

In order to meet the above mentioned requirements the MTPV is being extended for a pollution load corresponding to 4 million PEs. After completing the extension, the total wastewater from the city and several neighbouring municipalities will be treated at this plant (maximum dry and wet weather flows of 9 m^3/s and 18 m^3/s, respectively).

The existing high-load activated sludge plant is fully integrated into the extension design concept [2]. The design of the biological treatment process is based on a two stage activated sludge process concept with a newly developed flow scheme for nitrogen removal allowing optimal adaptation to the temperature variations during the year. During the design phase extensive pilot plant investigations of the treatment plant have been performed onsite [2] in order to prove this new concept well for full scale conditions. These pilot investigations were also used for the development of an adapted dynamic mathematical model (ASMV) and a specific aeration control strategy for the full-scale application. It has to be emphasized that sludge incineration [3] and the specific treatment concept play an important role with regard to nitrogen removal strategy. The full paper will concentrate on the scientific and technical considerations regarding the process development.

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The civil engineering works of the extension are already finished and the installation of the equipment is under way. The whole project will cost about 264 million €, of which about one half is spent on civil works. Simultaneously with the extension of the MTPV, intensive work is carried out in the sewer system to reduce combined sewer overflow discharges with the goal to transfer about 90% of the wet-weather pollution load to the treatment plant. For this reason new collectors are being designed and constructed and a new concept of wet-weather flow control in the whole sewer system is being implemented. The control system aims to achieve optimal use of the storage volume within the large collector sewers (~ 300,000 m^3).

2. Sewer system and receiving water protection

The catchment area of the sewer system is about 300 km^2. As can be seen from Figure 1, there are four important receiving waters, of which the Danube River is by far the largest, and about 10-20% of Danube flow is diverted to a sidearm Donaukanal, which joins the Danube again before the river leaves the area of the city. Most of the sewered area of Vienna on the right bank of Donaukanal is situated on clay and loamy soils, which results in high imperviousness of the area even in the Vienna Woods in the north and west of the city. There are many small creeks, which drain into the combined sewer network. They have very low flow during dry weather, but high peak flows during rain.

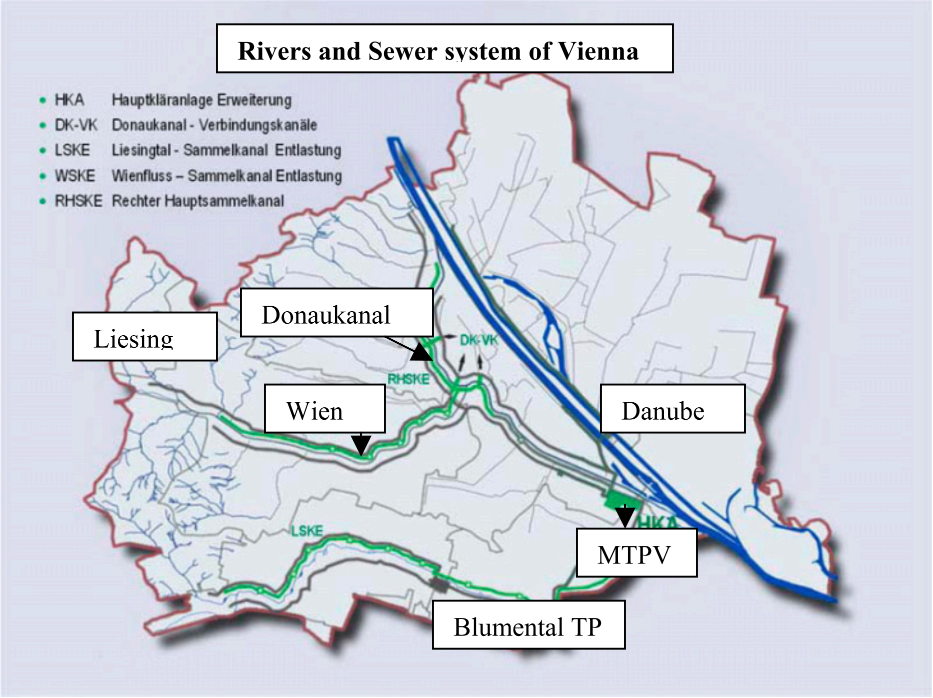


Figure 1. Urban rivers and sewer system in Vienna, Austria

There are only two other rivers or open creeks of importance. The Wien River (low flow 300 l/s, maximum high flow 600 m^3/s) crosses the most densely populated part of Vienna and Liesing Creek. Between these two streams there is a hill, which clearly separates the two catchments and their sewer systems. Both rivers can be classified as heavily modified water bodies. The whole Liesing catchment has a separate sewer system discharging to the creek at the Blumental Treatment Plant. This plant will be closed down after the extension of the MTPV has been finished. The sewer transporting the whole wastewater flow to the MTPV is part of the extension project. Along the other important rivers there are main trunk sewers on both banks. Especially those along the Wien River are heavily overloaded and a relatively great number of CSOs overflow frequently. A third sewer is under construction beneath the river in order to minimise CSOs and to enable realisation of at least ‘a good ecological potential’ for the river.

The catchment soils north of the Danube mainly consist of gravel with good infiltration capacity for rainwater. The whole region between the Donaukanal and the Danube had been a wetland before about 1870, when the actual riverbed was constructed in order to prevent flooding of the city and to gain additional area for urban development. For about 25 years now, the Danube in Vienna consists of two separate river beds. The right one is for normal flow, while the left one is divided into two man-made “ponds” during normal flow conditions. During high flow the gates are opened and this bed is used as a part of the river bed in order to increase the flow capacity. Between these two riverbeds there is an island, about 14 km long, which has become one the most important recreational areas for the citizens of Vienna.

Due to a hydropower station situated closely upstream of the confluence of the Donaukanal and Danube, nearly the whole reach of the Danube River within the municipality is dammed. Therefore both shores of the Island can be used for all kinds of water sports. As a consequence there is no CSO on the left bank of the Danube in order to prevent hygienic and eutrophication problems. The MTPV discharges to the Donaukanal and does not influence the reservoir of the power station. The discharge of the MTPV to the Donaukanal is located about 2 km upstream of the confluence with the Danube. Low flow in the Donaukanal is 50 m^3/s, and in the Danube about 800 m^3/s (mean flow is 1700 m^3/s). Therefore, the dilution ratio ranges from about 10 to 250, for mean dry weather wastewater flow of 6 m^3/s.

3. Legal requirements for wastewater treatment

Austrian Municipal Waste Water Treatment regulation is in accord with the homologue EU directive 271/91 [1] for sensitive areas but in addition it requires full nitrification throughout the year. The effluent discharge standard values to be met are shown in Table 1.

The values of Table 1 have to be met in daily flow proportional homogenised composite samples (>260 samples per year). Both self control and external control data have to meet this standard.

Table 1. Effluent standards for MTPV

Parameter	Dimension	95 percentile	Maximum	Yearly mean
BOD_5	mg/l	15	30	
COD	mg/l	70	140	
NH_4-N (T>8°C)	mg/l	5	10	
Total N (T>12°C)	% removal			70
Total P	mg/l		2	1

The MTPV is the largest treatment plant in Austria. It was designed for 2.9 million PEs and represents about 20% of the total Austrian biological treatment plant capacity. It went into operation in 1980. Actual Austrian and EU legislation requires the adaptation of the existing plant to the new regulations by 2005. The first attempts for the extension of the plant date back to the mid 1980s. After frequent changes of the legal requirements between 1985 and 1996 the final design and pilot plant investigation phase led to the actual implementation of the project, i.e., the construction of the extension to be finished by the end of 2004. With the start up of the extended plant, the second treatment plant in Vienna, the Blumental Treatment Plant (for 200,000 PEs) achieving >70% nitrogen removal and operated since 1969, will be closed down. Beyond 2005 all the wastewater from the City of Vienna and some of the surrounding municipalities will be treated at the extended MTPV.

4. Actual Situation at the Main Treatment Plant of Vienna

The Main Treatment Plant of Vienna went into operation in June 1980 as a one stage conventional mechanical biological treatment plant with a high rate activated sludge process [2]. The design capacity, based on 60 g BOD_5/PE/d, is 2.9 million PEs and the maximum design flow for primary treatment during wet weather is 24 m^3/s. The biological treatment is designed for a maximum flow of 12 m^3/s. The actual mean dry weather flow is in the range from 5 to 7 m^3/s. The activated sludge process was designed for a volumetric BOD loading of 3 $kg/m^3/d$, which corresponds to a sludge age of about 1.5 days. The process concept of the extension was developed with the goals of optimal use of the existing facilities and sustaining full uninterrupted operation of the existing plant until the start up of the extended plant.

5. Process concept of the extension

Figure 2 shows the layout of the extended plant, which is a special development of a two-stage activated sludge plant. The existing treatment plant, which becomes the future first stage, is shown in the upper part of Figure 2.

The original design data and operational results for the existing plant are summarised below:

Maximum design flow for primary treatment (wet weather):	24 m^3/s
Maximum design flow for biological treatment	12 m^3/s

Actual mean dry weather flow	6 m³/s
Volumetric BOD-load rate	3 kg/m³/d
Sludge age	~1.5 days
Actual permit BOD reduction requirement	≥70 %
Actual treatment efficiency for BOD removal	80-85%
New requirement since 2000 as an annual mean	1 mg TP/l

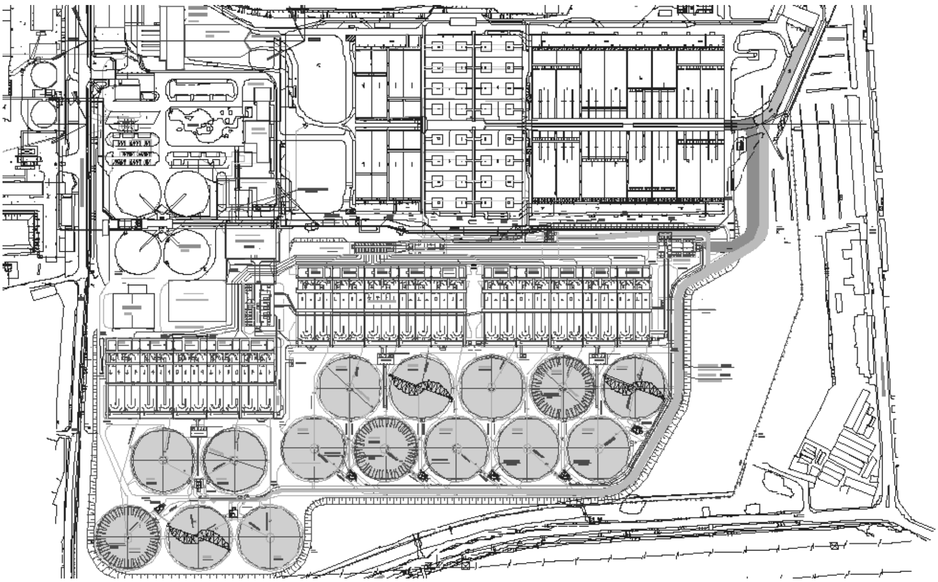


Figure 2. MTPV: existing plant (upper part) and plant extension (lower part, under construction)

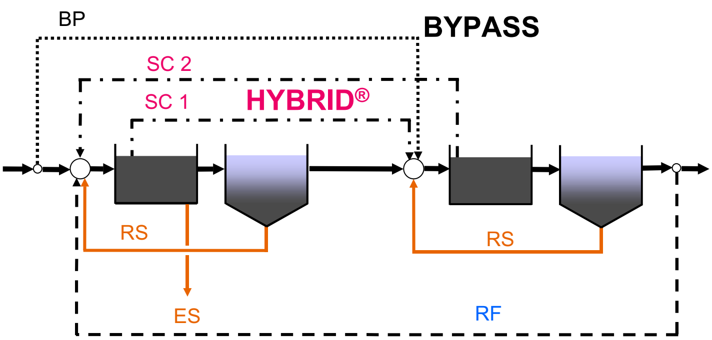
The basic design data for the extended plant are as follows:

Design load (2015)	4 million PE
Mean dry weather flow	7 m^3/s
Maximum design flow in dry weather (peak flow)	9 m³/s
Maximum hydraulic capacity (in wet weather)	18 m³/s

At the extended MTPV the total specific aeration tank volume of the plant is about 70 l/PE. For a single stage design according to the ATV A 131 guideline [4], the required specific aeration tank volume would have been in the range from 150 – 200 l/PE. As the raw sludge is incinerated next to the MTPV, the nitrogen load returned from sludge treatment is rather low. This strongly supports the chosen concept and enables high nitrogen removal efficiency with a low specific aeration tank volume.

In the new plant, under normal operating conditions, the excess sludge is withdrawn from the first stage only. The excess sludge of the second stage is pumped into the first stage. Sludge from the second stage, containing nitrifying bacteria, enters the first stage and thus enables nitrification in the first stage. The first stage is operated at a sludge age of about one day. If the sludge transfer from the second to the first stage would not be operated, this sludge age (1 d) would be too short to grow nitrifiers. The ammonia concentration in the first stage is at a level, which allows a maximum nitrification process rate. Figure 3 shows the flow scheme of the two-stage process modes.

A recirculation line will be installed, which delivers final clarifier effluent (containing nitrate) back to the first stage. In the first stage sufficient substrate for denitrification is available. The recirculation flow is adjusted automatically, so that the sum of incoming and recirculation flow is constant. This measure minimises hydraulic load variations to the plant, which minimises the loss of solids from the clarifier tanks [5].



BP	Bypass line	***SC 1***	Sludge circulation line 1	**RS**	Return sludge
RF	Recirculation flow	**SC 2**	Sludge circulation line 2	**ES**	Excess sludge

Figure 3. Flow scheme of the extended MTPV

For the two-stage extension of MTPV with regard to nitrogen removal two operational modes have been considered, the Bypass-mode and the Hybrid®-mode [6] indicated in Figure 3. In the Bypass-mode a portion of the incoming flow bypasses the first stage under all weather conditions. The Hybrid®-mode can be operated mainly during dry weather conditions, since the maximum hydraulic load to the existing biological stage of the treatment plant is limited to 12 m^3/s. If the influent flow to the plant exceeds this limit, the excess water bypasses the first stage and is directly sent to the second stage. The second stage is designed for a maximum flow of 18 m^3/s.

(1) Operation according to the Bypass-concept:

In the Bypass-concept a part of the plant influent flow is directly transferred to the second stage bypassing the first stage of the activated sludge plant. The bypass flow is operated within the range of 10 to 40 % of the total influent flow as a carbon source for denitrification in the second stage. Due to the bypass the nitrate containing recirculation flow from the final effluent to the first stage can be increased. In the first stage there is sufficient carbon substrate for denitrification.

(2) Operation according to the Hybrid®-concept [7]:

The Hybrid®-concept represents a two-stage activated sludge plant, comprising the exchange of activated sludge between the two stages. The sludge transfer between the two stages is carried out by means of two additional sludge circulation lines. Variation of the sludge circulation flow between the two stages allows operating the Hybrid®-mode within a broad range of operational characteristics. The exchange sludge flow never exceeds approximately 5 % of the plant influent flow. Hence, the exchange sludge flow does not impose an additional hydraulic load to the plant. The sludge withdrawn from the first stage by means of sludge circulation line 1 is an excellent carbon source for denitrification in the second stage.

In both operational modes the organic load transferred from the first to the second stage has to be controlled in order to maintain a sludge age sufficient for reliable nitrification in the second stage. The higher the nitrification rate in the second stage, the more volume of the second stage flow can be used for denitrification. The second stage has to provide complete nitrification in order to maximise nitrification capacity capable of handling ammonia peak loads.

Sludge containing nitrifying bacteria from the second stage is transferred to the first stage, where nitrification can take place despite the low sludge age. This measure induces a nitrifying population into the first stage. It is specific to the Hybrid®-concept but, as described above, the sludge circulation line 2 is operated in all modes utilising the benefits of this measure under all operational modes.

6. Pilot Plant Results

A pilot plant (scale 1:10,000) has been operated in order to investigate the performance and operating characteristics of the plant concept developed for the extension of the MTPV and to calibrate a mathematical simulation model [8].

The first stage which was operated with a sludge age of 1 to 1.5 days achieves more than 70% of the total carbon removal, 15% of the nitrification performance and 40% of the total nitrogen removal rate of the whole plant. The second stage was operated with a sludge age of approximately 6 days.

The temperature in the aeration tanks never fell below T = 12 °C, therefore the rigorous nitrogen removal requirements (Table 1) had to be fulfilled all year long. Despite of short periods of snowmelt conditions, when peak loads of urea containing road de-icing agents reached the treatment plant, the nutrient removal target could be achieved without any problems.

The Bypass-mode yielded a better nitrogen removal performance than the Hybrid®-mode, which was due to temperature effects and the increased nitrate return load to the first stage by means of the recirculation line. In the Bypass-mode a higher percentage of the produced nitrate is denitrified.

The bypass flow can also be adjusted to compensate for characteristic load changes. During the two years of operation of the pilot plant, a lower characteristic load has been observed on weekends. This is due to the stoppage of operation of the industry and trade and activities of urban dwellers outside of the city. The decreased load on weekends sometimes caused a shortage of substrate for denitrification in the second stage, which resulted in higher nitrate effluent concentrations. Increasing the bypass flow on weekends could be a possible measure to prevent this effect.

On the other hand nitrification can be operated more stably in the Hybrid®-mode than in the Bypass-mode, for two reasons: Firstly the dosage of primary effluent directly to the second stage imposed inhibitory effects on the nitrifying micro-organisms. A change to the Bypass-mode always caused an intermediate drop of the nitrification efficiency since the bacteria of the second stage had to adapt to the raw wastewater. Also an increase of the bypass flow after the system already had been adapted to the Bypass-mode resulted in a temporary drop of the nitrification efficiency. Secondly the SVI (Sludge Volume Index) of the second stage of the pilot plant increased during the Bypass operation periods [9] and therefore required a greater excess sludge withdrawal in the second stage leading to a reduction of the autotrophic biomass.

It can be seen in Figure 4, that in the Hybrid®-mode 85% of all ammonia effluent concentrations were below 2.5 mg/l, while in the Bypass-mode the respective effluent concentration was 7.0 mg/l. It has to be mentioned, however, that many special investigations have been carried out at the pilot plant and, therefore, the nitrification performance was not always kept at the optimum level during the experiments.

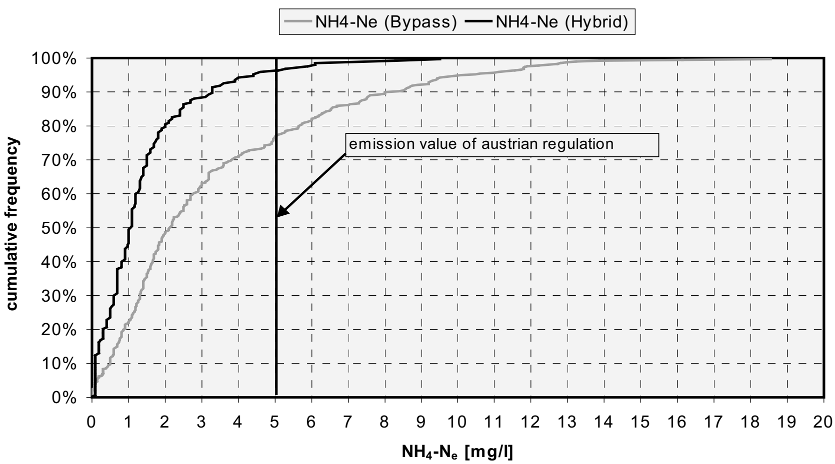


Figure 4. Cumulative percentage of effluent ammonia values of the pilot plant during Bypass- and Hybrid®-mode periods

7. Effect of Extension on Receiving Water Quality

The actual mean daily pollution loads discharged to the Donaukanal and finally to the Danube and the respective future loads are estimated as indicated in Table 2. The estimation is based on a mean loading of the MTPV of 3.5 million PE after the extension and 3.2 million PE at present. For the CSO it is assumed that about 6% of the actual raw wastewater production reaches the receiving water by direct discharge; for the future situation, after having put into operation a stormwater management system and the designed extension of the main sewers, a reduction to about 2% has been assumed.

Table 2. Pollution load discharges from the MTPV at present and after extension

		ACTUAL SITUATION		FUTURE SITUATION	
Parameter	Dim.	MTPV	CSO		
BOD_5 load	Mg/d	18	11	3	2
COD load	Mg/d	50	20	15	4
Ammonia N load	Mg/d	19	1	2	0.2
Total Nitrogen load	Mg/d	21	2	6	0.3
Phosphorus load	Mg/d	3 (0.6)	0.3	0.6	0.1

Since the loads of different parameters can not be added and it would be also incorrect to add BOD and COD loads from the effluent to those from CSOs, the positive effect of the extension with respect to the rivers cannot be expressed by one single numerical value, because the response of the receiving water is completely different. The global oxygen demand is one of the approaches to compress the treatment efficiency information. Global oxygen demand can be calculated by summarising the maximum possible oxygen consumption for the degradation of carbonaceous material (e.g., 90% of the COD load), for nitrification (NH_4-N*4.6) and the oxygen requirement for the oxidation of the autotrophic growth, i.e., algae production from the nutrient loads discharged to the receiving water (e.g. 80 gO_2/gP). Using such assumptions it can be concluded that the extension of MTPV and the CSO reduction program will lead to a reduction of the global oxygen demand by a factor of five. At the same time ammonia concentrations will drop by a factor of ten. Nitrogen removal will increase from about 40% to about 80%, which corresponds to a reduction by the factor of 3. From all these expected effects it can be predicted that the remaining existing detrimental effects of the actual discharges on the biological quality of the receiving waters (using the saprobic index) will disappear, and "close to natural" conditions will be achieved.

Recent investigations clearly show that a great number of endocrine substances, pharmaceuticals and household chemicals contained in the raw wastewater are only little affected by the actual treatment. Some of them will be reduced to very low concentrations after the extension [10].

8. Actual progress and costs

The aerial photograph taken in July 2002 (Figure 5) shows the stage of construction of the project "Expansion of the Main Sewage Treatment Plant of Vienna" after the first two years of construction [11].

In the foreground the completed first and second biological treatment blocks consisting of five lines can be seen. Each line consists of two aeration tank cascades and one circular secondary sedimentation tank. In the background the construction of the third and last treatment blocks can be seen, which was finished by the end of 2002. In the lower left construction area the completed construction of the compressor building, the new storage and workshop building, and the extension of the community building are shown.

Figure 5: Construction phase of MTPV in July 2003

The most intensive construction phase will last from the year 2002 until 2004 when the mechanical and electrical equipment will be installed. In 2002, for example, the first 64 m long double-arm scraper bridges were installed in the secondary clarifiers, which are visible in the right corner of the photograph (Figure 5). Until now all construction work has been completed according to the plan.

Construction started in the year 2000 and up to now around 117 million € have been invested - this is almost one half of the total construction costs amounting to 264 million €. Civil construction works account for ~88 million €, structural engineering to 12 million €, while mechanical equipment will cost ~64 million €. Another 44 million € will be spent on electrical, monitoring and control engineering. The remaining costs relate planning, construction supervision, project management, pilot plants, experts and control engineers.

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IMPROVING SEWAGE TREATMENT PLANT PERFORMANCE IN WET WEATHER

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1. Introduction

For the past few decades, since wastewater treatment plants have started to be built as a complement to urban sewer networks, wet-weather wastewater flows have started to create more or less significant operational problems to the treatment processes. It was just during the last decade that this problem started to be addressed with due consideration. In most cases, sewers and wastewater treatment plants have been designed on the basis of an empirical approach that takes into account a fixed level of permissible discharge into receiving water bodies. According to these empirical rules, the fraction of the incoming flow exceeding the maximum design flow (usually 2-6 times the average dry weather flow) is diverted to the receiving waters without treatment, in order not to "upset" the treatment processes. Still, during wet weather, wastewater collection systems deliver substantial amounts of stormwater runoff to wastewater treatment plants, creating conditions that range from process overload to bypasses of untreated or partially treated wastewater to process failure, in the worst case.

Wet-weather flows are generated in both combined sewer and separate sanitary sewer systems. While combined sewer systems are intentionally designed to convey stormwater and carry some small fraction of the stormwater flow to the treatment plant, separate sewer systems, although in theory designed to carry wastewater only to treatment plants, are also subject, through Infiltration/Inflow (I/I) phenomena, to deliver surprisingly substantial amounts of wastewater to the treatment works. I/I enters sewers either through structural defects or "natural" age-related deterioration of the structural characteristics of the system, or through inappropriate/illicit connections of storm drainage pipes.

The INTERURBA conference [1] held in 1992 at Wageningen (The Netherlands) dealt for the first time in a comprehensive fashion with the "integrated analysis" of the wastewater cycle, that is, the design and operation of sewer systems, treatment plants and receiving waters. Thus unavoidably, the conference also dealt with the problem of transient, wet-weather wastewater loads. The second INTERURBA conference, held in Lisbon in 2001 [2], provided some partial answers to the questions raised at the former conference. Because of both new regulations for water protection [3, 4] and an increased awareness of the effects of untreated overflows on

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receiving waters [5], the research aimed at improving the pollution control efficiency of the system as a whole has been conducted in intervening nine years. However, it resulted in situations, where the final connection (i.e., the receiving water component) was still missing from the analysis or based on strongly simplifying assumptions [6]. More research and applications are still therefore needed in this area.

The most frequently reported measures to control wet-weather flows in the system are local infiltration, source control, use of storage basins, use of local (satellite) treatment and real-time control. Some specific applications involving treatment plant upgrade or operational enhancement have also been experimented with and reported in the literature, as discussed in this paper.

2. Effects of wet-weather flows on treatment processes

The majority of Sewage Treatment Plants (STPs) is designed to treat some stormwater, by being designed to treat typically 2-6 times the average dry weather flow, depending on local standards/regulations. The excess hydraulic capacity available at the STP, when compared to its design flow, is therefore rather small compared to the total potential inflow into a combined sewer system during wet weather. Hence, full hydraulic loading of the plant could be caused even by a low intensity rain event, with duration greater than the time of concentration of the combined sewer catchment. As a consequence, the maximum capacity of the plant can be reached even at an early stage of a significant storm. When this happens there is no other choice than to bypass the treatment process for the rest of the event or to delay its arrival at the plant. In addition to limiting the fraction of the total influent that undergoes treatment to the above mentioned ratios, through flow diversion/ bypasses, a frequently adopted solution consists in storing excess flow in the sewer network or in specially designed off-line storage structures, by means of appropriate controls. Such controls can be both physical and numerical, in order to maintain the state of treatment processes as steady as possible and within a physically-sustainable capacity.

As the practice of in-sewer storage of wet-weather flows is getting more widespread as a means of controlling undesired CSO discharges, it becomes apparent that this also leads to problems at the end-of-the-pipe treatment plant, such as [7, 8]:

- Increase in first (foul) flush loads, due to the increased sedimentation of grit and organics in the pipes during in-sewer retention periods;
- increase in screening, grit and sludge masses, even in excess of those already high values that can be observed during a normal first-flush event without flow storage control;
- longer periods of plant operation at full hydraulic load after the storm event, due to long drain-down times of the stored combined sewage;
- disruption of treatment process efficiency for carbon, nitrogen and phosphorus removal, due to waste septicity, or changes in COD and nitrogen levels, caused by prolonged in-sewer residence times; and,
- reduced biomass performance due to waste composition changes and/or low substrate/high flow loading periods after the event.

In the absence of in-line/off-line sewer flow storage, during wet weather, the following other problems can also occur during wet weather load conditions [9, 10]:

- Biomass washout from the aeration basins to the clarifiers by higher flow rates; this could eventually lead to an overall loss of solids from the system, if clarifiers cannot sustain the separation process at these increased flow rates;
- slower biomass growth due to a combination of increased influent flow and higher ammonia concentrations, often accompanied by a temperature drop;
- increase of both ammonia and nitrates in the effluent, also due to load flushes of dissolved pollutants (especially ammonia) that arrive to the treatment plant in advance of the flow peak [11], and to increased contents of oxygen (in normal flow conditions) and higher dilution of incoming wastewater;
- decreased efficiency of Bio-P removal processes, especially in the presence of high nitrate and oxygen levels, and of reduced acidification of the incoming wastewater in the sewer;
- unstable biomass settling characteristics, which may lead to high levels of suspended solids in the effluent, caused by diminished SRTs;
- creation of density currents within the secondary clarifier, due to influent flow's increased energy, that may cause loss of solids via the effluent;
- sludge blanket rise in the clarifier due to the increased mass transfer rate of biosolids from the AS tank to the clarifier. If this is beyond the capacity of the settling tank, it could start to affect directly the effluent quality and eventually lead to excess solids loss in the effluent, up to process failure conditions.

While the final effects of wet-weather flows can be different from plant to plant, there are a few established types of intervention that may be adopted to improve the performance of STPs during critical conditions, such as those above mentioned.

3. Improving STP performance during wet weather

3.1 CLARIFICATION

A treatment plant typically includes three separate settling/clarification process steps: grit removal, primary settling and secondary settling. All of these processes are very sensitive to hydraulic loading, and thus to the hydraulic loads associated with wet-weather flows.

In order to improve the solids removal efficiency of clarification in overstressed plants, attempts have been made to increase process capacity: these mainly consisted in the modification of clarifier flow regimes, and chemical addition [12]; dissolved air flotation and filtration have also been considered as supplemental process for the treatment of such flows.

In general, as the clarification process is usually very sensitive to hydraulic transients, care should be taken in the sizing and combination of pumps in plants working under artificial head gradients. Pumping stations should be designed in such a way that no on/off transient shock waves are generated when the plant is subject to wet-weather inflows, as well as during dry weather conditions, because these can cause upsets in the status of the units and eventually of the entire system.

3.1.1. Physical Modification

Physical modification of clarifiers consists in plate and tube settlers, under different possible configurations. Essentially, these are formed by rows of inclined parallel plates, or tubes (Figure 1); as the liquid flows upwards between their walls, the solids settle on the inclined surfaces and slide into a sludge collecting hopper at the bottom of the unit (Figure 2). The presence of the plates has the effect of increasing the equivalent surface area of the basin, allowing higher overflow rates and thus increasing the clarification capacity up to tenfold. Some problems in the application of these devices in wastewater plants (they were originally developed for use in drinking water treatment plants) have been reported with respect to plugging by biological growth, floatables, oil and grease.

The use of special proprietary systems, such as the DELREB® system [13] and the Parkson Lamella® clarifier [14] has also been investigated in full-size and pilot plants. The DELREB® system combines plate settling with chemical addition, and consists of two coagulation chambers, one flocculation chamber and a plate settler with a sludge thickening zone. Coagulants used are ferric chloride and polymer. A study conducted at a treatment plant in France showed that this unit has the capacity to operate at solids loadings up to 26 kg/m^2/h with a 80% TSS removal efficiency; this corresponds to a hydraulic loading of up to 2000 m^3/m^2/d considering an average influent concentration of 300 mg of TSS/. The unit could also operate, at lower levels of efficiency, at hydraulic loadings up to 3600 m^3/m^2/d.

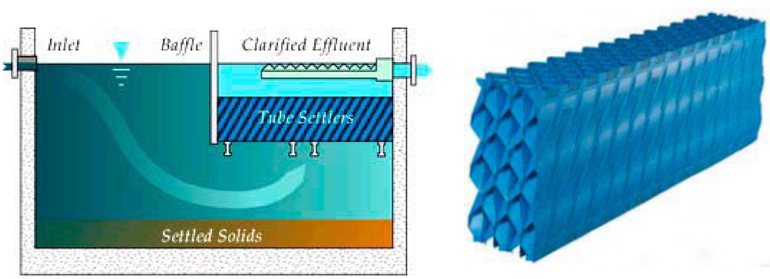

Figure 1. Example of tube settlers installation (left) and particular of the material

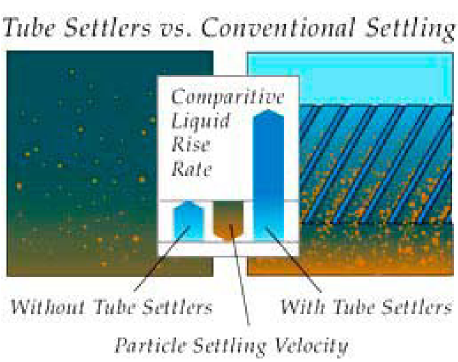

Figure 2. Solids behaviour in a clarifier with and without plate/tube settlers

The use of tube settlers was also reported at another facility in France with a somewhat erratic performance (ranging from 0-90% SS removal, with an average of 54%) attributed to plugging and inadequate installation.

These systems usually work under optimal conditions when accompanied by the addition of flocculating chemicals, however, a study conducted on a Lamella® clarifier application with flocculation showed a start-up time for the unit of about 2 hours, which renders it somewhat inadequate for intermittent operation [15].

It should be noted that stress testing of the existing basins should be carried out prior to implementing modifications of the traditional units. Stress testing implies the application of increasing hydraulic, organic and solids loads to the existing units in order to identify their "failure points" in the operating "plane" SOR/SLR (Surface Overflow Rate/Solids Loading Rate). Failure can be defined for primary clarifiers as the loading that results in the failure of the secondary system due to organic overload; secondary failure can be defined as the loading that results in a violation of discharge permit limits for solids and BOD (COD) [16].

3.1.2 Physical-chemical processes

In addition or as an alternative to the physical modification of the units, the solid particles themselves can be modified in order to promote their aggregation into larger/heavier clumps that offer better settling properties, by addition of appropriate chemicals, most commonly ferric chloride and polymer, but also aluminium sulphate and iron oxides [17]. In all these cases, optimum dosages have to be identified by experimental procedures (jar tests) on the specific types of wastewater.

Recently, micro-sands have also been used as micro-carriers in ballast-coagulation of colloidal particles, with the aim of increasing the settling velocity. The process consists of the addition of a coagulant to the influent, micro-carrier (sand) and coagulant aid in a downstream mixing chamber, followed by flocculation and sedimentation, and is usually denoted with the generic term BFP (ballasted flocculation process). With such processes, however, sludge quantities produced increase considerably, together with sludge handling costs. Actiflo® (Krüger) is currently the only commercial system that uses recycled microcarrier sand (Figure 3), while the conceptually similar DensaDeg® uses recycled sludge as a ballast agent.

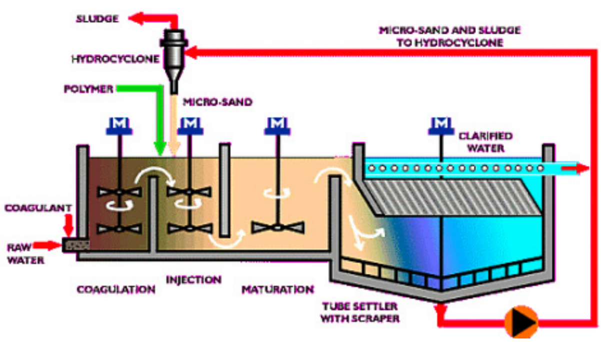

Figure 3. Scheme of a BFP process (Krüger)

Pilot and full-scale applications of microsand-based systems have shown reductions of up to 85-95% of TSS, 60-80% of BOD and 85-95% of Total P at overflow rates up to 10 times greater than the conventional ones. Sludge-based systems have shown a more erratic efficiency in pilot applications and usually require start-up times of up to 4 times longer than the BFP systems, however, they are able to produce a much thicker sludge (up to 4% solids compared with 0.2% in a BFP).

3.1.3 Other processes

Dissolved Air Flotation (DAF) has been used for the separate treatment of wet-weather wastewater flows in the suburbs of Paris [18]. The facility provided separate, one-step, treatment of wet-weather combined wastewater with 90% efficiency of TSS removal from the first minute of operation. An application in the US with the goal of improving wet-weather performance of a conventional STP in Oregon is documented by the EPA [19]. DAF was applied to primary treatment with the conversion of the secondary treatment to contact-stabilisation mode. DAF efficiency reached over 70% in TSS and BOD removal, with modest capital costs.

Filtration applications have also been reported as complements to other processes (DAF, conventional) [17, 18], with mixed results (good removal efficiencies, but also the risk of premature clogging).

3.2 BIOLOGICAL PROCESSES

Separate and combined stormwater flows exhibit wide composition variability from storm to storm as well as within a given storm. The effect of initial concentrations on biological process performance was qualitatively described in Section 2 of this paper and can be evaluated directly by finding the order of the reaction as well as the rate constant, however, it is expected that these will also vary depending on instantaneous conditions. Temperature variations during the event will also affect the efficiency of biological processes.

3.2.1 Detection of process upsets

In order to avoid "bottleneck" conditions, accurate information concerning the actual treatment capacity of the plant must be available. As the capacity of any single process changes continually with time, depending on operating and flow conditions, key parameters should be monitored on-line. Some properties of the system (such as biomass characteristics and sludge settleability) are prone to vary on a medium to long-term basis (hours to days), and may continue to affect process performance after the systems returns to "normal" inflow conditions, without the possibility of compensating for temporary overloading by operational means. Others can be more readily controlled if an abnormal situation is detected in time.

The beginning of an overload condition in nitrification processes can be detected immediately by observing rising ammonia concentrations in the nitrification volume. This can be done on-line, with sampling time of about five minutes or less [20, 21, 22]. This allows sufficient time for a proactive operational response, as the biological system's reaction time is in the order of hours. Similarly, denitrification performance could be improved by external addition of a carbon source to the wastewater should the nitrate/carbon ratio fall out of the optimal range. All these parameters can be measured on-line (Figure 4), at very high frequency (up to one measurement every 10 seconds) by using cost-effective submersible UV-VIS spectrometers, thus allowing effective control.

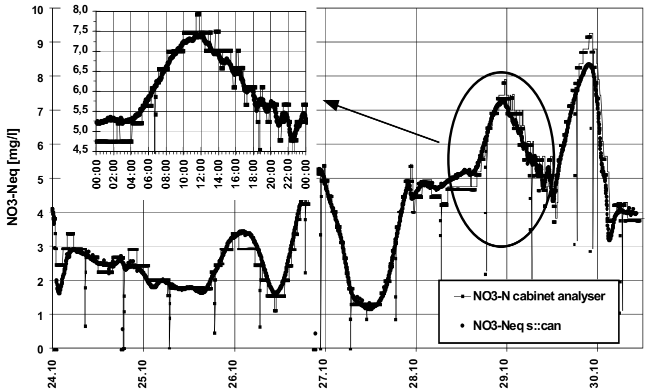

Figure 4. Continuous measurement of nitrate in WWTP effluent with on-line sensor (thick line), compared to discrete measurements by a cabinet analyser (Courtesy of S-CAN GmbH, Vienna)

3.2.2 Hydraulic regime

Another critical assumption is the type of mixing regime that takes place in the treatment reactor. The two limiting cases are plug flow, wherein the particles simply "queue" through the reactor, and complete mixing, wherein the incoming particles instantaneously mix with the reactor liquor. In the case of sudden transient peaks, the flow regime of the tanks could be modified by altering significantly the hydraulic residence time (HRT) of the unit, beyond what could be expected just by the variation in flow, due to internal short-circuiting. Hydraulic verification of the performance of the units under variable flow loading conditions could point out the need for modifications/improvements of the tanks. Usually the use of modified inlets or baffles/diverters is sufficient to restore the functionality of the units.

Modification of the unit's flow regime during wet-weather events can be useful to improve the overall plant performance; the utility of step feed operation of plug-flow AS basins for reducing secondary clarifier sludge blanket height and improving effluent quality have been demonstrated in several instances.

Under step feed, the influent wastewater feed point is adjusted from the head section of the step-feed aeration basin to the points at sections downstream (at half-length or at third- or quarter points) of the basin. As usually the recycled sludge is fed at the head section of the tank, this shift increases the solids concentration in the first aeration compartment, creating a sort of "sludge reserve" that is not pushed along the tank as the flow is now entering from more downstream points. This consequently reduces the solids load to the secondary settler. Depending on how this strategy is implemented, this mode of operation can actually be used to shift solids from the clarifier, from where they are continuously extracted, to the AS basin. The required modifications for implementing step feed include a distribution canal and motorised gates that can be manipulated from the plant control room. On-line instrumentation should be available for monitoring mixed liquor suspended solids (MLSS) distribution, dissolved oxygen (DO) concentration, settler sludge blanket height and effluent suspended solids concentration. With normal sludge settling characteristics, step feed has been shown to increase plant storm flow capacity considerably by as much as 50% more than under "normal" operating conditions, and to substantially reduce the frequency and quantity of bypasses required.

High-rate operation of STPs during and following wet-weather events is an important option to be evaluated as a part of the overall stormwater management program for combined and separate systems that are affected by I/I. It is in theory possible to model the expected performance of these systems by means of process simulation software and conducting continuous simulation of the effect of wet-weather flows [8]. High-rate operation of an STP during wet periods and periods with high I/I, for example due to seasonably high groundwater table, appears to be a very attractive option to consider, however, not much research has been done on this problem. Only a few citations on the results of attempting to model the dynamics of STP operation during high flow periods can be found in the literature, and intrinsic model limitations, derived from both model development hypotheses and knowledge of dynamic phenomena, must be carefully taken into account.

3.3 EXCESS FLOW FACILITIES

Excess Flow Facilities (EFFs) can be used to supplement the treatment capacity of a STP during high flow periods. The flow in excess of the maximum allowable flow in the weakest component of the system (the so-called "bottleneck" of the plant) is then bypassed and, rather than being discharged untreated to the receiving water body, it is diverted to the EFF. Therefore, an EFF acts as a safeguard to conventional plant systems by giving the operator additional control over the amount of flow entering the treatment plant.

An EFF generally employs a purely physical treatment, usually clarification. For example, an EFF may consist of a First Flush Tank (outer "ring" tank) and an Excess Flow Clarifier (centre tank). When a storm event occurs and the plant capacity is reached, wastewater begins to fill the outer tank until it becomes full. Should the excess flow continue, the wastewater then "spills over" into the Excess Flow Clarifier where the heavier solids are removed by sedimentation.

Other EFF units may employ DAF [18], high-rate filtration, etc., and can be complemented by disinfection of partially-treated of untreated water, where concerns for specific uses of the receiving water exist (e.g., bathing, recreation).

4. Real-time STP control and on-line monitoring

The performance of a STP at any time is restricted by its weakest component at that instant, creating a "bottleneck" situation in the system [23]. When not properly recognised, bottlenecks may cause partial or total failure of the treatment process (Figure 6). The time sequence of set points of all the possible regulators (pumps, gates, valves, etc.) and constraints in a control system is termed *control strategy*. Control strategies may have different final objectives, including the following:

a. Minimisation of untreated overflows. In this type of control, flow storage, release and overflow are manipulated to minimise discharge of untreated combined sewage into receiving waters;

b. Stabilisation of treatment processes and effluent quality. Under this type of control, the operator maintains process parameters in the stable range and targets effluent concentrations at or below standards. Depending on the type of process, key parameters must be identified, and appropriate control algorithms implemented, acting on a manipulated variable. Flow exceeding the STP capacity is bypassed;

c. Minimisation of total pollution loads. Both effluent and overflow quality are considered to reduce as much as possible the total load into receiving waters;

d. Economy-based control (e.g., reduction of energy consumption for pumping and/or oxygen supply, reduction of chemical additions, reduction of "non-compliance" fines).

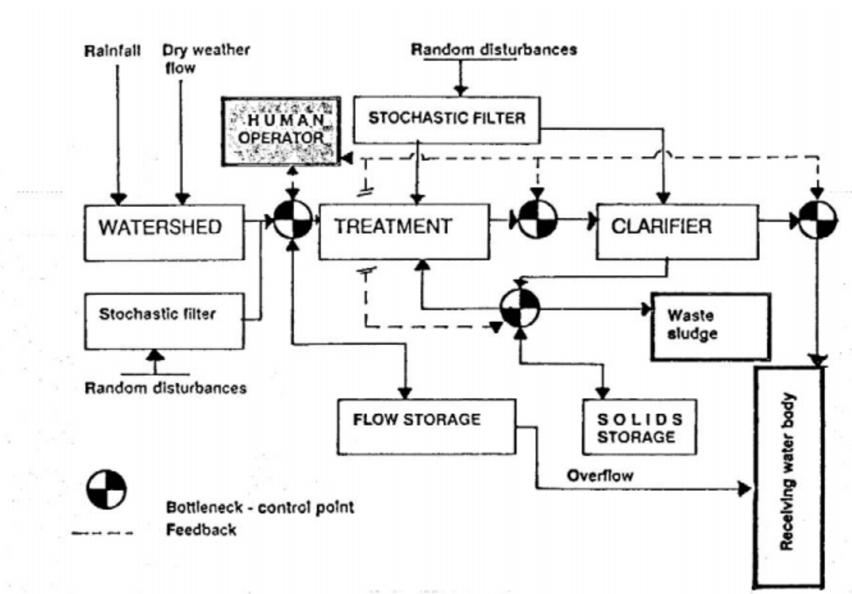

Figure 6. Schematics of WWTP with bottlenecks/control points (from [23]). Components of random nature within the system are emphasised

Performance of a control strategy as a measure to reduce wet-weather impacts on STPs is strongly dependent on the specific wastewater system and the type of pollution problem in the receiving water.

In addition to on-line monitoring, on-line modelling should be also used in these cases. To date, there are several models that represent deterministically wastewater treatment processes. Among these models, the most used is probably the "Activated

Sludge Model No. 2" (ASM2) [24]. Others have been developed by individual researchers for their use [25].

Usually, results can prove quite satisfactory, even under "extreme" conditions; however, some caution must be taken, *in primis* by understanding the simplifications of the real system that were hypothesized in their development phase. While their representation may be "true" during normal operation, it might not stand to proof during wet-weather conditions. Unusual flows, short-circuiting, and localized turbulence may also invalidate some basic underlying assumptions on which the model equations are based. At best, a careful evaluation of the performance of the plant can lead to temporary adjustment, or "special calibration" of the model for exceptional events.

The adoption of mixed deterministic-stochastic (*grey box*) models may improve in these cases the forecasting performance for use in on-line control [26].

5. Conclusions

Treatment plants may be subject to significant process upsets during extreme flow conditions. The type and extent of the upset will vary depending on plant configuration, processes, and gradient-duration of the load transient. Better utilisation of STPs with respect to the attainment of receiving water quality goals can be achieved by implementing appropriate process modifications and/or additions and by adopting sensible control strategies, supported by an initial "flexible" design of the facility. Adequate on-line monitoring capabilities are needed in all these cases.

6. References

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UPGRADING SEWAGE TREATMENT PLANTS FOR HIGHER PERFORMANCE BY OPTIMISING REJECT WATER TREATMENT

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1. Introduction

The intensification of the anaerobic sludge digestion process yields higher biogas production and lower sludge production, but also induces a negative change in reject water composition – the increase of ammonia nitrogen concentration. If such nitrogen rich wastewater is rejected to the mainstream of a wastewater, the ratio COD/N decreases and efficient biological nitrogen removal starts to be problematic.

Methods of the separate treatment of reject water, which are intensively studied, can be divided into three groups:

- Controlled discharge in mainstream of wastewater
- Physical and chemical treatment
- Use of reject water for bioaugmentation, and
- Biological treatment

Controlled discharge enables to optimise efficiency of biological nitrogen removal in the mainstream of wastewater treatment. This way peak loads and peak effluent concentrations can be eliminated. It was found, that main controlling parameter is not the organic or nitrogen loading rate of the biological system, but the ratio COD/N. Reject water should be discharged when the ratio is high enough.

A stripping of ammonia or struvite precipitation are the most frequent methods of physical and chemical treatment of reject water.

In order to increase nitrification capacity of an activated sludge system without the necessity to increase reactor volume the bioaugmentation method can be used. When dosing ammonia rich reject water into an oxic tank placed in the return sludge flow stream nitrification capacity of the plant can be significantly elevated [1].

Many wastewater treatment plants have already introduced the separate biological pre-treatment of the reject water [2,3,4]. The specific character of the reject water (pH, $N\text{-}NH_4^+$, COD/N) offers new alternatives for biological treatment.

The most promising processes are:

- Deammonification, which means the transformation of ammonium into elemental nitrogen under oxygen limitation, without using organic carbon source. Ammonium is

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partly oxidised to nitrite that is denitrified to N_2 by the rest of ammonium [5,6] and
- Nitrogen removal via nitrite [7,8].

The described study was focused on nitrogen removal via nitrite, which is easier to start and operate.

The advantages of the nitrogen removal via nitrite are:

1. the reduction of oxygen demand for the nitritation (25 %), in comparison with the nitrification, and
2. the reduction of organic substrate demand for denitritation (40 %), in comparison with the denitrification.

Nitritation	$NH_4^+ + \mathbf{1{,}5\ O_2} \rightarrow NO_2^- + 2\ H^+ + H_2O$
Nitrification	$NH_4^+ + \mathbf{2{,}0\ O_2} \rightarrow NO_3^- + 2\ H^+ + H_2O$
Denitritation	$2\ NO_2^- + \mathbf{6\ H^+} + 6\ e^- \rightarrow 2\ OH^- + N_2 + 2\ H_2O$
Denitrification	$2\ NO_3^- + \mathbf{10\ H^+} + 10\ e^- \rightarrow 2\ OH^- + N_2 + 4\ H_2O$

Nitritation and denitritation processes were studied at the laboratory scale to evaluate the optimal technological conditions for the stable and high rate nitrogen removal during the pre-treatment of the reject water.

2. Materials and methods

The reject water of the Prague central wastewater treatment plant was used for experiments; its quality is illustrated in Table 1.

Table 1. Reject water composition

Parameter	Unit	Average	Max	Min
pH	-	7.88	8.42	7.18
COD	(g/l)	1.16	2.72	0.39
COD(soluble)	(g/l)	0.80	1.81	0.12
TSS	(g/l)	0.32	1.35	0.15
$N\text{-}NH_4^+$	(mg/l)	1247	1513	943
CODsol / $N\text{-}NH_4^+$	(g/g)	0.61	1.19	0.29
Neutralisation capacity $_{(pH\ 4.5)}$	(mmol/l)	95.1	107.1	76.6

Average COD of primary sludge was 75.7 g/l (3.65 g/l in soluble form), TSS concentration was 67.1 g/l, with a 66.6 % organic fraction.

The laboratory experiments were carried out in a two-stage sequencing batch unit with a first stage oxic reactor and a second stage anoxic reactor (Figure 1).

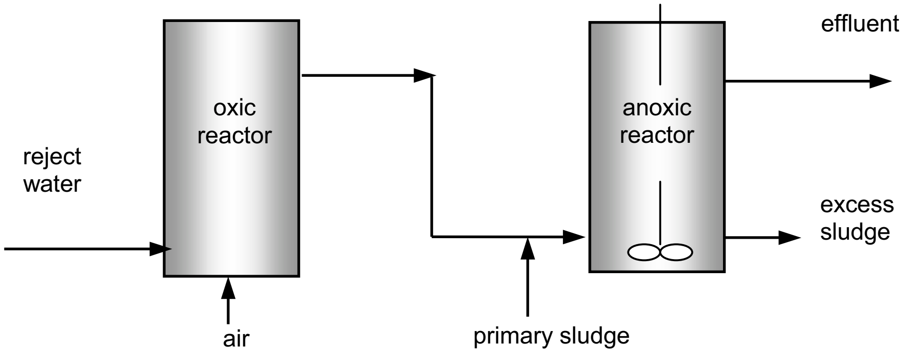

Figure 1 – Experimental set-up

The primary sludge was added as a source of organic carbon in the second stage [9]. The operational cycle was 6 hours.

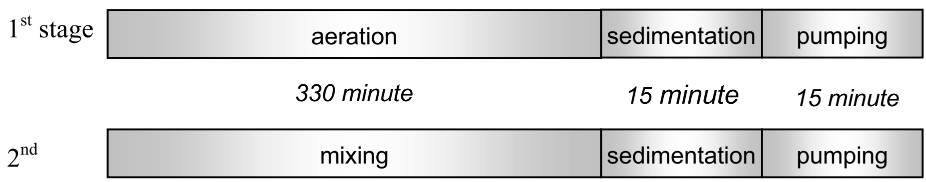

Figure 1 – Operational cycle of both oxic and anoxic reactors

3. Results and discussion

3.1 NITRITATION

The activated sludge of the Prague central wastewater treatment plant was used for the start-up. To achieve efficient nitritation a change of microbial population was necessary. The selection pressure facilitating the change was developed by the following factors: (i) low COD/N ratio (ii) high pH, and (iii) limited oxygen concentration

Formation of nitrates was almost suppressed in several days and more than 90% of oxidised compounds of nitrogen were represented by nitrites.

The volumetric loading rate of the oxic reactor by nitrogen was increased step by step from 0.2 to 0.8 kg/m^3.d. The excellent performance of the nitrite production and the efficient inhibition of the nitrate production are shown in Figure 2.

The results achieved allow following data evaluation concerning the factors affecting selective inhibition of nitritation.

3.1.1 $NH_3 + NH_4^+$ concentration

The sum of $N\text{-}NH_3$ and $N\text{-}NH_4^+$ concentrations of treated wastewater was about 1200 mg/l and it was also operational concentration at the beginning. During start-up of nitritation process the concentration decreases and during the final period of operation, when nitritation was almost completed, $N\text{-}NH_4^+$ concentration was much lower. The variation during the operation cycle of SBR was approximately 600 mg/l at the start of cycle and several tens of mg/l at the end. NH_4^+ and especially free ammonia NH_3 concentrations were certainly considerable inhibiting factors during start-up, but later other factors probably play a more important role (Figure 2). Furthermore, in the case of inhibition by free ammonia, the micro-organisms can be acclimated according to [10,11].

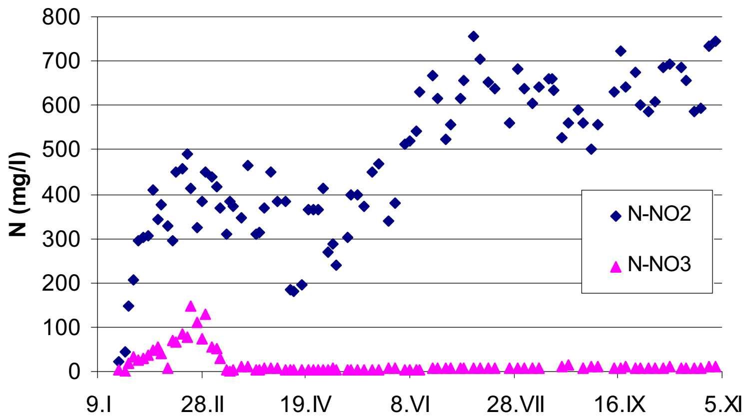

Figure 2. Nitrate and Nitrite production

3.1.2 COD/N ratio

The COD/N ratio of wastewater generally determines the microbial population of activated sludge. At low COD/N, a higher share of N-oxidizing bacteria can be expected, at the expense of heterotrophic bacteria. The only change found during the period with higher COD/N was a higher TSS concentration in the nitritation reactor. A relation between COD/N and nitritation was not observed.

3.1.3 pH

The importance of pH value during the nitritation process seems to be crucial. The pH controls the distribution of NH_4^+/NH_3 and NO_2^-/HNO_2 as well. During the start-up the concentration of NH_3 is high owing to high pH. The adjustment of pH to 8.5 results in more than a 10 % share of $N\text{-}NH_3$. On the other hand a decrease of pH to 7.0 implies that NH_3 is absent. Similar changes induce the change of pH in the system of NO_2^-/HNO_2. At pH 7.5 practically no nitrous acid is present. If pH decreases to 6, the share of HNO_2 increases to 0.2 %. At a steady state operation without pH adjustment the variation of pH was from 8.0 to 6.6 in one operational cycle of the nitritation reactor. During the period with pH control the variation interval was even wider and shifted to higher pH levels (8.6 – 7.0).

3.1.4 Dissolved oxygen concentration

It was shown during the experiments that limited oxygen concentration limits not only the generation of nitrate, but also the efficiency of nitrite formation. In circumstances of our experiments, inhibition of nitrite oxidisers by other factors seemed so strong that dissolved oxygen concentration was not appropriate as a control parameter.

3.1.5 $HNO_2 + NO_2^-$ concentration

Nitrite and HNO_2 concentrations are the important factors affecting the efficiency of NH_4^+oxidation to nitrite. It was found during experiments that low pH (6.0 – 6.5) and high concentrations of nitrite (700-1200 mg/l) cause the inhibition of further nitritation. At such conditions, HNO_2 concentration was about 1-1.5 mg/l. According to the results of Anthonisen et al. [12] more less the same concentration causes inhibition of *Nitrobacter* (nitrite oxidizer). Consequently it could be assumed that the inhibition of ammonia and nitrite oxidisers starts at a similar HNO_2 concentration or in our experimental conditions the inhibition of nitrite oxidisers may have started at even a lower concentration.

3.1.6 Sludge age and temperature

Experiments were performed at a laboratory air temperature of 21 ± 1 ^{o}C and due to low sludge production, at a very high sludge age (> 30 d). In spite of these facts nitritation did not take place.

The reason for different conclusions drawn in various studies regarding the factors affecting nitrogen removal via nitrite can be found mainly in different concentrations of N-compounds (NH_3 and HNO_2 especially) and in the first place, in different actual concentrations that the micro-organisms are in contact with [13].

3.2 DENITRATATION

When the applied primary sludge/reject water ratio was higher than 5:100 in the anoxic step, the complete removal of nitrates was ensured.

The removal of ammonium was stable at the end of the start-up procedure, in the range from 60 to 80% (see Figure 3). In fact the efficiency was 2-3% better, because a small amount of ammonium was added during the dosing of the primary sludge.

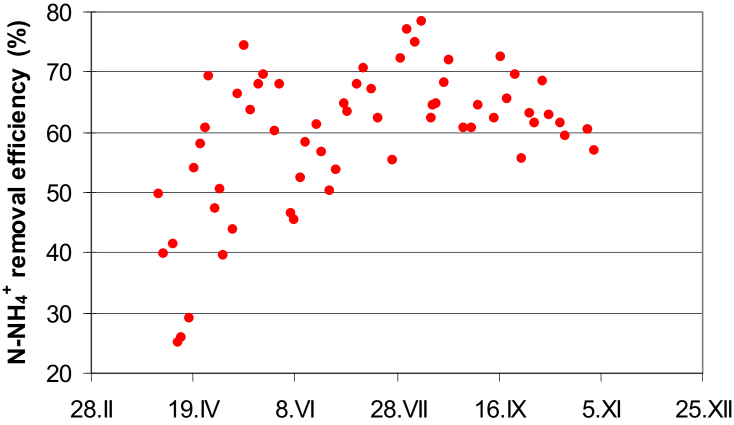

Figure 3. The efficiency of ammonia nitrogen removal

There was highly active and well settling biomass in both nitritation and denitritation reactors, with average SVIs (sludge volume index) of 98 and 48 ml/g respectively.

Tables 2, 3 and 4 summarise the main results of the first phase of experiments. The average treatment efficiency of 73% (expressed for $N\text{-}NH_4^+$) was achieved at an exceptional nitrogen loading rate of 0.32 kg/m^3.d.

Table 2. Average results of the1st stage

Parameter (unit)	Value
Hydraulic retention time (d)	1.5
Loading rate (nitrogen) (kg/m^3.d)	0.64
COD removal (%)	38.2
Efficiency of nitritation (%)	58.9
Efficiency of nitratation (%)	0.5

Table 3. Average results of the 2nd stage

Parameter (unit)	Value
Hydraulic retention time (d)	1.5
Vol. ratio primary sludge : reject water	0.05
Loading rate ($N\text{-}NO_2^-$) (kg/m^3.d)	0.38
Efficiency of denitritation (%)	98.9
$N\text{-}NH_4^+$ removal efficiency (%)	12.5

Table 4 - Average results for the total system

Parameter (unit)	Value
Hydraulic retention time (d)	3.0
Temperature (^oC)	21
Loading rate (nitrogen) (kg/m^3.d)	0.32
Loading rate (COD) (kg/m^3.d)	0.29
$N\text{-}NH_4^+$ removal efficiency (%)	73.0

COD removal (%)	20.3

From an economic point of view it is very important that at the achieved treatment efficiency (about 70 %) additions of neutralisation agents are not required.

4. Conclusions

Separate treatment of the nitrogen rich reject water improves the COD/N ratio in the mainstream activated sludge process and consequently improves the efficiency of the biological nitrogen removal in sewage treatment plants.

The composition of the reject water is suitable for the use of new biological processes like nitritation/denitritation.

Primary sludge can be successfully used as the carbon source for the denitritation, which can be complete and controlled by the primary sludge / reject water ratio.

Treating the reject water with the extremely low $COD_{soluble}/N\text{-}NH_4^+$ ratio of 0.64, without an external carbon addition (considering the primary sludge as an internal source), at the volumetric loading rate up to 0.32 kgN/m^3, the average nitrogen removal efficiency above 70% was achieved.

5. Acknowledgement

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UPGRADING WASTEWATER TREATMENT PERFORMANCE

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1. Introduction

In the Czech Republic, 70% of the domestic sewage is collected and treated in existing wastewater treatment plants (WWTPs). There are about 900 existing WWTPs. In order to comply with the EU legislation and the European urban wastewater directive, existing WWTPs need to be rehabilitated or upgraded. Considering the high number of existing WWTPs this could result in high costs.

The development of designs and subsequent construction of new medium-sized and large-sized WWTPs in the Czech Republic has finished. In exceptional cases, old WWTPs will be relocated to new sites. The last major large-size WWTP servicing the town of Děčín has been completed and future development of wastewater treatment in the Czech Republic will concern the refurbishment and updating of existing WWTPs. Biological wastewater treatment using the activated sludge process has become the prevailing technology and this trend will certainly continue in the near future. The technological line itself is or must be modified as enforced by the valid legislation, and consequently, systems with enhanced nitrogen and phosphorus removal must be introduced.

The general objectives of updating and refurbishing of WWTP´s can be described as follows:

- Substantial increase in the efficiency of mechanical-biological treatment, mainly in terms of biogenic elements – nitrogen and phosphorus.
- Updating and increasing the efficiency of separation in secondary clarifiers.
- Substantial reduction in energy consumption of the treatment process.
- Minimising the environmental effects of the water and sludge lines.
- Enhancing the ease of operation and improvement of the operation culture.
- Use of superior newly installed equipment leading to reduction in repair and maintenance costs.
- In order to obtain optimal operating conditions, introduction of higher degree of automation and technological process control.
- Minimising the total operating costs (reduction in the number of operators, reduction in electric energy consumption, reduced charges for discharged pollution, and reduced costs of repair).

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2. Methods of enhancing the efficiency of nitrogen and phosphorus removal

Two basic technologies are used almost exclusively:

a. Circulating activated sludge process with a front-end anaerobic zone
The original civil structures at the WWTPs are usually utilised as anaerobic tanks. The circulating activation tanks are newly constructed structures. The secondary clarifiers undergo upgrade of the mechanical-technological equipment and are utilised according to their sizes or are constructed as new civil structures.

b. The R-AN - D-N system
This is a traditional Czech technology with sludge regeneration zones, anaerobic zone, denitrification zone and nitrification zone in rectangular tanks. This relatively segmented system achieves high usability of the existing tanks system, where the original activation tanks, secondary clarifiers, pipe collectors etc., are used for the individual sections. Part of the activation volumes is then newly constructed, usually with new up-to-date secondary clarifiers.

3. Aeration systems

When reconstructing medium and large-sized WWTPs in the Czech Republic, the use is made almost exclusively of fine-bubble aeration with membrane elements. Surface aerators are not used in municipal WWTPs. The fine-bubble aeration technology is much more efficient in terms of oxygen transfer (the normal use of oxygen in the air is 5 - 6%/m) compared to other technologies, the oxygen feed rate in the activation system can be easily regulated and the price is reasonable. The service life of the membranes based on EPDM is more than 8 years, and given proper operation there are no problems with clogging of the elements and their performance does not drop rapidly (by 10 - 15 % in 8 years). It proves that it is not necessary to acquire systems enabling removal of the aeration grids during operation. Fine-bubble aeration with membrane elements works very well in the circulating activated sludge process controlled by the start-stop system. Its use is conditioned by high quality blowers and corrosion-resistant pipe distribution system; the activation system must be equipped with degassing sections before the inlet to the secondary clarifiers.

4. Final clarifiers

The operation of wastewater treatment systems with the state-of-the-art technological modifications of the activation process in this country as well as abroad demonstrate the key role played by the secondary clarifiers in achieving high quality of effluents. The best activation process cannot produce a good quality effluent unless the secondary clarifiers meet their purpose. Therefore, the reconstruction work always includes the updating of secondary clarifiers. Normally, the original unsuitable clarifiers are utilised for other purposes (as denitrification or anaerobic zones) and new clarifiers, having prevailingly a round shape with radial flow and flocculation zone and other new components, are installed. The secondary tanks are provided with the following new components:

- Flocculation chamber, made fully of concrete, or with a central steel column and a steel or plastic flocculator.
- Special dividing elements ensuring distribution of the mixed liquor in the flocculation chamber.
- This solution eliminates problems with the generation of floating scum cap in the flocculation chamber (clearing or use of a closed flocculation cylinder).
- Optimised outflow of the mixed liquor from the flocculation chamber leading to the most uniform possible distribution of the mixed liquor flow conveyed to the secondary clarifier.
- New types of systems removing floating impurities also eliminate problems related to excessive production of poorly removable scum scraped off the surface of the secondary clarifiers.

5. Minimising emissions affecting the plant surroundings

In recent years, the WWTPs in the Czech Republic started to take measures leading to reduction of the impacts on plant surroundings. Two trends can be observed:

- Enclosing of civil structures having extreme production of aerosols (screw pumping stations, screen buildings, grit chambers). For the time being, other structures of the water line are not enclosed. In exceptional cases, deodorizing filters are installed.
- Covering of dewatered sludge dumping sites, or the use of sludge storage tanks.

6. Diagnosis and process control

Wastewater processes do have some unique features:

- The daily volume of wastewater treated can be large.
- The disturbances in the influent are enormous compared to most industries.
- The influent must be accepted and treated; there is no option for returning it to the supplier.
- The concentrations of nutrients are very small, even challenging the sensors.
- The process depends on micro-organisms, which have a definite mind of their own.
- The reliable separation of the effluent from the biomass is challenging.
- The value of the product in the marketplace is remarkably low.

7. Process or plan goals

- To meet effluent discharge requirements. In most cases average values are specified over a period such as a week or month. This involves determining average operating conditions which will maintain average effluent quality close to and within regulatory requirements.

- To achieve a good rejection of disturbances. This involves manipulating operating conditions about their average values, in order to compensate for the effects of varying influent conditions and to maintain effluent quality constant. This ensures environmental responsibility and keeps the effluent within the maximum discharge limits, where these exist. It can also be very costly to allow poor effluent to be discharged, and then try and compensate for it by achieving extra good quality effluent during a compensating period.
- To optimise operation to minimise the operating costs. The principal components of operating costs are generally interest on the capital expenditure, air and chemicals, and operation and maintenance manpower. Interest can be minimised on a long-term basis by maximising the utilisation of equipment, thus deferring and reducing capital expenditures.

8. Case studies

8.1 WWTP MODŘICE

The WWTP in Modřice treats domestic sewage and industrial wastewater from local industrial plants. It was constructed as a mechanical-biological plant including sludge disposal and energy recovery. Wastewater treatment is ensured by gradual separation of coarse and fine suspended solids followed by biological treatment degrading soluble matters and colloidal dispersed substances. The construction of the whole original plant took 10 years and the WWTP was put into operation in 1960. In spite of all the modifications carried out in the course of time, the plant could not meet all limits. As a result of legislative changes at the beginning of the 1990s, greater attention started to be paid to the nutrient removal – nitrogen and phosphorus, which have a major adverse impact on the quality of surface water in the case of unfavourable dilution ratios in the receiving water.

The reconstruction and extension of the wastewater treatment plant is one of the largest infrastructure investments in the City of Brno. The main target of this project is to reconstruct the existing civil structures at the plant, which are outdated or in a poor state (above all, the mechanical treatment and sludge disposal), to construct a new biological stage, to increase the capacity of the sludge dewatering plant and to construct a sludge drier that will be the first of its kind in the Czech Republic. Furthermore, an automated operation control system will be implemented and the European effluent standards will be met. This mainly relates to the removal of nitrogen and phosphorus, which was not achievable by the original WWTP.

Table 1. Capacity of the newly reconstructed WWTP

Population equivalent	513,300 PE
Total design daily flow	137,000 m^3/day
Average hourly flow	5,708 m^3/hour
Maximum dry weather flow	7,600 m^3/hour
Maximum wet weather flow	15,200 m^3/hour

Table 2. General scope of work

Water line		**Sludge line**	
Stormwater tank	new	Sludge thickening (primary and biological sludge)	new
Gravel and sand traps	reconstructed	Sludge dewatering (centrifuges)	new
Primary sedimentation tanks	6 reconstructed 2 removed	Sludge drier	new
Pumping stations	reconstructed new	Digesters	reconstructed
Activation tanks	new	Storage tanks	converted
Secondary sedimentation tanks	new	Energy recovery system	new
Blowers	new	Gas holders	new
		Biofilters for air treatment	new

Due to the complexity of the technological processes and the effort to achieve the state-of-the-art wastewater treatment management, all parts of the plant will be equipped with corresponding automation and regulation systems including a central control room.

8.2 JIHLAVA WWTP

The Jihlava WWTP is a classic mechanical-biological wastewater treatment plant with average-loaded activation process and anaerobic sludge stabilisation. As the plant experienced problems with increasing pollution as early as in the 1980s, the aeration system was replaced in 1994 along with the installation of a fine-bubble aeration system. At present, the consulting company Hydroprojekt has developed a design of the overall reconstruction of the WWTP. We present this WWTP as an example of combination of the aforementioned technologies, i.e., circulating activated sludge process and the R-AN-D-N system.
Following the primary sedimentation, wastewater will be conveyed by gravity to a

newly arranged activation system consisting of the following:

- Regeneration tank (original activation tank) divided into anoxic and oxic zones.
- Anaerobic tank (original secondary clarifier, diameter of 30 m) with a separated anaerobic sector.
- Circulating activated sludge process with simultaneous nitrification and denitrification as two lines of a circulating activation tank.
- Aeration system with combined aeration (fine-bubble aeration plus slow-running horizontal mixers).
- New round, secondary clarifiers, with a diameter of 30 m, and water depth at the wall of 5.8 m. The tanks will be equipped with scum scraping, inlet and flocculation barrier of a new design (closed surface level) including a baffle cone, clarified water outlet trough, scum board, overflow weir and deflector.

The updated sludge disposal will include mechanical thickening of excess sludge using a centrifuge, correct homogenisation of primary and thickened sludge, and mechanical dewatering of sludge. A dewatered sludge dumping site will be constructed. A design dealing with incineration of dewatered sludge will be developed. The projected hydraulic capacity is expressed as a flow rate of Q_{24} = 12891 m^3/d and a load corresponding to 99,917 PE (person equivalents).

8.3 NOVÉ MĚSTO NA MORAVĚ WWTP

The WWTP is designed for the final capacity of 18,700 PE. The wastewater treatment plant is designed as a mechanical-biological plant connected to a combined sewerage system with the following components: pre-treatment, low-loaded circular aeration tank, secondary sedimentation, sludge storage tank, centrifuge and a device for measuring the amount of discharged water.

A new facility for the interception of incoming coarse solids (gravel and sand entering into the WWTP via the main sewer) has been designed as part of the WWTP upgrading. New machine-raked racks with a press will be installed in the main inflow trough. The second set of machine-raked racks will remove load from the existing rack technology. The existing storm tank pump will be replaced with a new mud pump to ensure failure-free pumping at the bottom. An end flap will be installed at the outflow from the storm tank. A stainless steel sand separator will be added to the detritus sand. An underground sump will be established at the WWTP for the delivery of outside sewage with a volume of 30 m^3 furnished with a sludge pump and a mixer. A new aeration tank with fine-bubble aeration will be established with air blowers. The blowers will be located in a new building with proper access. Frequency converter control will be installed. A new aeration system will be installed, ensuring sufficient supply of air for the oxidising of contaminants both in the new and existing tanks. The activation is designed for a sludge load of 0.05 kg BOD/kg.d and sludge concentration of 4.0 kg/m^3. A new technology for machine scum collection will be installed in the final settler to eliminate floating scum from the final settler. An additional final settler will be built. The type and dimensions of the secondary clarifiers are important factors influencing the efficiency of the WWTP. Their proper functioning is decisive for the quality of the final effluent. It is stated that each 10 mg.l^{-1} of leaking activated sludge deteriorates the basic indicators of the discharged wastewater quality (BOD by 5 $mg.l^{-1}$, N_{total} by 0.6-0.8 $mg.l^{-1}$, $P_{total.}$ by up to 1 $mg.l^{-1}$). The functionality and efficiency of the secondary clarifiers will also affect the potential recycling rate of the return sludge. In sludge management, the capacity of the existing storage reservoirs will be extended by building two additional tanks. Reconditioning of the machine sludge dewatering (centrifuge) has been designed with the aim of increasing the dry residue contents to 25 to 30%. The dewatered sludge will be disposed at a nearby compost plant in Bystřice nad Pernštejnem where it will be transformed into industrial compost.

8.4 HUSTOPEČE WWTP

This WWTP is designed for the final value of 9,900 PE. The wastewater treatment plant is designed as a mechanical-biological plant connected to a combined sewerage system with the following components: intake chamber, screw pumping station, septic water tank, fine screens, sand trap, stormwater tank, circular activation tanks, secondary clarifiers, sludge dewatering and measurement of the volumes of discharged wastewater.

The existing design of the mechanical-biological wastewater treatment plant will remain unchanged. In the past the WWTP fairly successfully dealt with removal of organic contamination, and was capable of a high degree nutrient removal by nitrification and de-nitrification. Intake chambers and screw pumps will be covered. Intake station for septic water will be built and mechanical pre-treatment will be upgraded by installing separators and sand washers. Equipment allowing the rinsing of sediment from stormwater tanks will be installed. Oxygen needed in the aeration process will be introduced by means of an air blower and a fine-bubble aeration system. The activation is designed for a sludge load of 0.05 kg BOD/kg.d and a sludge concentration of 4.0 kg/m^3. Dosing of ferric salts for phosphorus precipitation will be provided in the outlet of the activation tanks. The secondary settling tanks are circular with bottom and surface scrapers. The sludge storage tanks are designed with sludge mixers, and with zone removal of sludge water permitting thickening of the sludge. De-watering of sludge will be achieved by means of belt press. The de-watered sludge will by hygienised. The aim of the proposed measures is a significant improvement in the effectiveness and reliability of the WWTP in removing organic contamination, and especially removing nutrients (N and P) in accordance with the ČR and EU legislation, and also a general stabilisation of the operations of the WWTP.

9. Concluding Remarks

- Reconstruction and upgrading is always understood to be a complex measure.
- It is not always possible to fully utilise previously built facilities in updating since these were not included in the overall reconstruction.
- It is recommended to take advantage of reduced volumes of wastewater and the resulting redundant capacity of tank volumes.
- Newly constructed civil structures are those that replace structures with insufficient volumes (activation tanks), or unsuitable shapes or sizes (secondary

clarifiers).

- Application of the traditional systems R-D-N and R-AN-D-N, as well as the new application of the circulating activated sludge process with combined aeration as the oxic stage. The main reason is the possibility of a high degree of oxygen transfer regulation.
- When changing the layout of a WWTP, a proper design of the hydraulic profile of the WWTP is a difficult task. It is necessary to perform detailed calculations and this problem must not be underestimated.
- Despite some repeatability, each reconstruction of a WWTP is an original job and must be dealt with specifically both in terms of calculations and the technical solutions themselves.

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BULGARIAN POLICY AND ACTIONS FOR MUNICIPAL WWTPs CONSTRUCTION AND UPGRADING

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1. Introduction

Bulgaria is among the countries with the smallest water resources in Europe of about 2,400 m^3 per capita per year. Currently only about 42% of those resources are useable, because of pollution caused by domestic, industrial and diffuse (agricultural runoff) sources, among which the point pollution sources prevail. With the definite trend of increasing water consumption, the water deficit is going to become one of the most severe national problems. That is why wastewater treatment is recognised by the government as a priority, which is very important for the society with its significant social, economic and ecological impacts.

Currently over 98% of population of Bulgaria is connected to centralised water supply systems, 66.8% is served by sewer systems and only 40% is served by wastewater treatment plants (WWTP). The total number of the settlements served by sewer systems is 277, including 167 towns (70.2%) and 110 villages (2.1%). The total length of sewers equals 48.5% of the total length of streets in towns and only 0.6% of the total length of streets in villages. The sewer networks are usually of the combined type and consist of concrete pipes. Significant infiltration is observed in almost all of the sewer networks currently in operation, which changes drastically the wastewater quality parameters and creates serious problems in implementing advanced wastewater treatment technologies and biological nutrients removal in particular.

At the moment 61 municipal wastewater treatment plants are in operation in the country, 50 of which use conventional secondary treatment technologies. They serve 67 settlements and about 40% of the country population. Only 26 towns with population over 10,000 (out of 97 in total) have wastewater treatment plants among which only 13 WWTPs serve relatively large towns with population over 50,000 (out of 28 in total). Forty-one existing wastewater treatment plants urgently need rehabilitation or reconstruction or upgrading, with implications for the associated sewer networks as well.

Bulgaria has to meet specific economic, social and environmental requirements in the pre-accession period before joining the European Union, which is expected to take place by 2007. Environmental and infrastructure related issues are among the

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ones of high priority for the country. Their successful solution is based primarily on the implementation of the relevant legislation and the best available technologies (BAT).

National policy, programmes and actions in the field of wastewater treatment are subjects of this paper, which is focused on the technological challenges created by the current National and EU legislation in the field of Urban Waters. General types of the wastewater treatment plant process flow charts applied in the country are discussed briefly in the light of the best available technologies, along with the relevant organisational problems and financial parameters and resources.

2. Legislation background

The process of harmonisation of the Bulgarian National legislation with EU directives is almost completed in the fields of environmental protection and water resources having taken the relevant EU directives into account and the Water Framework Directive 2000/60/EEC [9] in particular. The subject of this paper is related to the following legislation background:

- Water Act [10]
- National Programme for Priority Construction of WWTPs for the Settlements with Population Over 10,000 population equivalents (PE) [4] (called hereafter the National Programme for WWTPs)
- Implementation Program for Directive 91/271/EC Concerning Urban Waste Water Treatment in Bulgaria [2] (called hereafter the Implementation Programme)
- National Programme for Construction of Sewer Networks and Main Trunks to the WWTPs for the Settlements with Population Over 10,000 PE [3] (called hereafter the National Programme for Sewerage)
- Ordinance for Establishing Sensitive Zones [5] (called hereafter the Sensitive Zones Ordinance).

Water Act [10] is a framework law for the National policy in the field of water resources management. Detailed regulation of different aspects of water resources management is performed through 16 regulations, associated with the Water Act. Regulation No. 6 Wastewater Effluent Quality Limits [6] in particular is closely related to the subject under consideration. Among others it includes all the requirements of the EU Wastewater Treatment Directive 91/271/EEC [1] concerning the wastewater effluent quality limits both in sensitive and in less sensitive zones with respect to nutrient (N and P) concentrations and the receiving water vulnerability to eutrophication. Requirements of the Directive 91/271/EEC on the implementation of the best available technologies (BAT) in sewer network design and construction are also included in Regulation No. 6.

In the National Programme for WWTPs [4], 36 settlements are included in Stage I (i.e., the period up to 2007) as the first priority of the WWTPs construction, completion or reconstruction, for which 325.5 million EUR are necessary. In this Programme altogether 104 municipal WWTPs are considered during the period spanning until 2015, among which 71 are scheduled for construction, 10 for completion and 23 for reconstruction.

Some issues in the National Programme for WWTPs are updated by the newly adopted Implementation Programme [2], which deals with planning construction of WWTPs in settlements with population between 2,000 and 10,000 inhabitants. To fulfil this programme 309 small WWTPs are envisaged to be constructed by 2015, in addition to updating some of the 121 WWTPs in the settlements with population over 10,000 to become operational by 2011.

In the National Programme for Sewerage [3], 36 settlements are included in Stage I (period up to 2007) as the first priority for construction of the sewer networks and main trunks connected to the WWTPs in a compliance with the National Programme for WWTPs. In the former Programme 128 settlements are included altogether, for which sewer network and main trunk construction costing 1,493 million EUR is required. The investments are distributed over three stages as follows: 559.4 million EUR in Stage I, 492.9 million EUR in Stage II and 440.7 million EUR in Stage III.

The Ordinance for Establishing Sensitive Zones [5] is issued by the Ministry of Environment and Waters (MEW) according to the requirements of Regulation No. 6, associated with the Water Act and it is in compliance with the requirements of Directive 91/271/EEC. This Ordinance establishes areas along stream reaches and the associated aquifers, belonging to the relevant river basins, which are designated as “sensitive” or “less sensitive” according to a set of criteria. The later include the vulnerability of the receiving surface waters (river reaches, lakes and seas) to eutrophication and/or the probability of nitrogen concentration limit violation in groundwater used for drinking water supply. Receiving waters belonging to a “sensitive zone” impose more stringent wastewater effluent quality limits and, therefore, call for more complicated and expensive wastewater treatment technologies. That is why the establishment of “sensitive” and “less sensitive” zones in the river basins is an important legislative act of the government with specific economic and ecological consequences. Since the main Bulgarian rivers flow out of the country or into the Danube River, forming a part of the country border, this issue became internationally important with respect to a number of transboundary river basins, shared between Bulgaria and its neighbouring countries, mainly Greece and Turkey, as well as to the Danube river basin and the Black Sea. According to the Water Act, four river basin regions are established in Bulgaria: *Northern*, covering all the Danube tributaries on the Bulgarian territory; *Eastern*, covering all the Bulgarian rivers draining into the Black Sea; *South-Western*, covering the transboundary river basins of the Struma (Strimon) and the Mesta (Nestos); and, *Southern*, covering the transboundary river basin of the Maritza (Evros, Merich) and their large tributaries the Tundja and the Arda. The last two river basin regions belong to the Aegean Sea basin. Currently the Ordinance for the Sensitive Zones Establishment defines all the *Northern* and *Eastern* river basin regions (belonging to the Danube and therefore to the Black Sea catchment, respectively) as “sensitive zones”, while both “sensitive” and “less sensitive” zones are assigned to the *South-Western* and *Southern* river basin regions.

3. Financial resources

Recent estimates show that about 3,500 million EUR are needed for the development, upgrading, reconstruction and rehabilitation of the urban water infrastructure (UWI) in Bulgaria. For the development, upgrading, reconstruction and rehabilitation of the sewer networks and WWTPs only, the required sum is about 2,200 million EUR. The state and the municipalities are unable to secure this sum of money at present or in the nearest future. That is why the support from the international financial institutions and participation of the private sector (foreign investors in particular) is of vital importance.

The following sources of finances are specifically or provisionally considered by the government and municipalities in this respect [3, 4]:

- State Budget (SB)
- Enterprise for the Management of Environment Protection Activities (EMEPA)
- European Investment Bank (EIB)
- European Bank for Recovery and Development (EBRD)
- PHARE (EU programme supporting institutional and economic development of the Central and Eastern European (CEE) countries after the political changes in 1989)
- ISPA (EU fund supporting the infrastructure development of the Central and Eastern European (CEE) countries during the period of their pre-accession to the EU)
- Private Investors (PI) – International companies participating in the urban water infrastructure (UWI) management and development
- Bilateral financial agreements between the Bulgarian government and other European countries or banks.

The state budget (SB) contribution to the sewer networks and WWTPs construction, reconstruction and rehabilitation is limited to 282 million EUR by 2010, which is about 8% of the necessary investments for the urban water infrastructure (UWI) development. During the period from 1998 to 2001, the SB contribution was only 79,300 EUR. It is envisaged that 88.8 million EUR will be spent from the SB in 2003, on reconstruction and rehabilitation of several municipal WWTPs.

The Ministry of Finance (MF) is a principal of all the grants, loans and SB subsidies allocated to the UWI development in Bulgaria. The Ministry of Environment and Water (MEW) and the Ministry of Regional Development and Public Works (MRDPW) are managing the UWI projects implementation and the utilisation of the relevant funds. For this purpose, a joint Project Management Unit (PMU) was created by the two ministries. The sewer networks and WWTPs design projects are subject to discussions and approval by the Higher Technical Expert Council (HTEC) at the MRDPW and/or the Higher Ecological Council (HEC) at the MEW. The Managing Board (MB) of the Enterprise for the Management of Environment Protection Activities (EMEPA), headed by the Minister of

Environment and Water is responsible for the EMEPA financial resources allocation for UWI projects and, in particular, for support of sewer networks and WWTPs development.

The European Investment Bank (EIB) through its specialised funds supports mainly projects related to the sewer network and WWTP construction and rehabilitation. In the period from 1977 to 2001, EIB has invested 703 million EUR in the Bulgarian UWI. It is going to invest further 28 million EUR in the next two years for construction of the WWTP in the Town of Haskovo (210,000 PE) in the transboundary Maritza River basin.

The European Bank for Recovery and Development (EBRD) supports mainly municipalities. It has invested 31 million EUR in Bulgaria up to now, supporting the concessionaire of the Sofia UWI. A part of this sum was used for the Sofia WWTP reconstruction.

The PHARE programme includes several target-oriented programmes, among which the INTERREG programme is devoted to bilateral projects between CEE and EU countries directed towards solving their transboundary problems, including those related to the environment and infrastructure. Through the PHARE programme a grant of 17 million EUR was allocated for reconstruction of the Sofia WWTP, which is almost completed. About 5 million EUR are allocated for Bulgaria by the INTERREG III programme during the period 2003–2004 for reconstruction of the WWTP in the Town of Razlog in the Mesta (Nesos) transboundary river basin. About 6 million EUR are allocated through the PHARE programme, supplemented by 2.3 million UR from the SB for construction of 3 other municipal WWTPs (in the towns of Madan, 9,000 PE; Rudozem, 16,000 PE; and, Zlatograd, 24,000 PE, respectively) in the Arda transboundary river basin.

The ISPA programme provides grants, covering up to 75% of the cost of specific infrastructure projects in the CEE countries during their pre-accession period. The remaining 25% is covered by the governments of the beneficiary country. In Bulgaria, these grants are shared equally between UWI projects and transportation infrastructure (TI) projects. In the period 2000–2001, four UWI projects were funded by ISPA with a total sum of 66.2 million EUR. Currently 43 million EUR are allocated for construction of two municipal WWTPs in the towns of Stara Zagora (348,000 PE) and Dimitrograd (297,000 PE), respectively, in the Mariza (Evros, Merich) transboundary river basin. Another 12.5 million EUR are allocated for construction of WWTP in the town of Gorna Oryahovitza (637,000 PE) in the Danube River basin. In the next few years, ISPA will allocate about 1,418 million EUR for construction of another 11 WWTPs in towns around the country with 1,755,800 PE in total.

In the beginning Bulgaria applied to ISPA for funding packages for WWTPs only, neglecting the associated sewer networks rehabilitation and development. Recently this approach was recognised as wrong, since groundwater infiltration into the deteriorated sewer networks, made of concrete pipes, results in highly diluted wastewater influents to the WWTPs. This in turn does not allow adequate design and performance of these WWTPs, as well as implementation of some advanced biological wastewater treatment technologies for nutrient removal, which are indispensable for the areas designated as “sensitive zones”. Recent Bulgarian applications for ISPA funding include packages of municipal WWTPs and the associated sewer networks.

The private investors (PI) participation in the UWI projects in Bulgaria is expected to contribute to the vast amount of the investments needed. Its contribution is envisaged to reach the sum of about 2,000 million EUR, which is 57% of the required total sum of 3,500 million EUR. The first significant step in this direction is the concession treaty between the Sofia municipality and the UK private company United Utilities for the UWI management in the period 2001–2026. For this purpose, a new joint public trading company, named Sofiyska Voda (Sofia's Water) was created. One of the priorities of the new company is reconstruction of the Sofia WWTP which is now being implemented with the help of a grant of 17 million EUR from the PHARE programme. For reconstruction of the high-rate anaerobic digesters of the Sofia WWTP, additional 4.6 million EUR was invested by Sofiyska Voda in 2003.

The Bulgarian government also signed contracts and utilised financial support for the UWI development through some bilateral agreements, such as the one with the German bank KfW, which provided a loan of 12.7 million EUR. Based on a bilateral agreement with the Swiss government named "Environment for Duty", the Bulgarian government transferred some financial resources in 1998 to support reconstruction of the WWTP of the City of Plovdiv (1 M PE). The Danish government and the Danish Environment Protection Agency (DEPA) provided grants for construction of WWTP in the towns of Obsor and Biala (0.9 million EUR), as well as for construction of WWTP in the Town of Troyan (1.1 million EUR). Another grant was provided by DEPA for rehabilitation of the sludge treatment facilities at the WWTP of the City of Varna (1.53 million EUR).

4. Implementation of the National Programme for WWTPs

The availability of the necessary financial resources is without any doubt of primary importance and a prerequisite for the UWI projects implementation. As shown above, there are no serious problems in this respect in Bulgaria at this stage. Actually the successful implementation of the UWI projects now depends entirely on the activities of MRRB, MEW and their joint PMU. In this respect, however, there are significant problems resulting in a very low percentage of the available financial resources utilisation, which for the ISPA programme is not more than 20% by now. The reasons for this are to a great extent subjective, rather than objective.

Design and tendering procedures according to the National legislation and requirements of EU and the relevant International banks have already started for 19 municipal WWTPs out of the 36 scheduled for construction and upgrading in the first stage of the National Programme for WWTPs [2, 4]. Currently only two of these 19 WWTPs (in the towns of Stara Zagora and Dimitrovgrad) in the Maritza transboundary river basin have passed all the necessary procedures and are in the construction phase. Actually the WWTPs at the towns of Dimitrovgrad and Shumen are in the last phase of construction, which is to be restarted after a long period of stagnation. Currently 17 municipal WWTPs are in different stages of the design and tendering procedures, which are expected to be completed in the next few months.

Reconstruction and upgrading of the Sofia WWTP is already under way. However, the appropriate legal and technical preparatory work is completed for construction or upgrading of the remaining 16 municipal WWTPs, scheduled for the first stage of the National Programme for WWTPs, in violation of the timetable established initially for this Programme. Among them, six WWTPs are scheduled for upgrading.

Concerning the technical aspects of the WWTP construction and upgrading it has to be pointed out that their technological flowcharts and basic technological parameters have been approved by the Higher Technical Expert Council (HTEC) at the MRDPW before establishing the "sensitive" and "less sensitive" zones in the domestic river basins. After issuing the Sensitive Zones Ordinance in July 2003, it appears that most of the WWTPs do not meet the relevant requirements for the nutrients removal or meet them only partially. In addition, almost all of the WWTPs technological flowcharts (with one exemption indicated in Table 1) envisage chlorination for disinfection, which is forbidden in the EU. In this respect disinfection with ultra-violet (UV) rays is recommended, which was planned at one WWTP only.

Obviously a general review of the relevant WWTP technological flowcharts is necessary on the basis of the best available technologies (BAT), for which additional time and financial resources will be necessary, as well as adequate actions by the competent institutions.

5. Technologies applied at the municipal WWTPs

Technological flowcharts of most of the municipal WWTPs, which are currently at different stages of the tendering process, include secondary treatment and some of them even tertiary treatment, as indicated in Table 1. The only exemption is the WWTP in the Varna district of Asparuhovo, where after primary treatment the wastewater is to be pumped to the main WWTP of the City of Varna for secondary treatment.

At two WWTPs listed in Table 1 (Shumen and Blagoevgrad) conventional aerated tanks were planned, resulting in BOD removal only, which will be not adequate for the requirements of the relevant receiving waters classified as sensitive zones (Table 1). At other four WWTPs listed in Table 1, the biological reactors included in the technological flowchart were designed to operate in the extended aeration mode only, which will ensure the necessary level of BOD removal and nitrification, but will not be sufficient for the required level of nutrient removal. At other two WWTPs simultaneous coagulation was included which will ensure only phosphorus (P) removal in the aeration tanks along with BOD. Only five WWTPs were designed to achieve both nitrogen (N) and phosphorus (P) biological removal. However, in one of the last cases, for instance in Haskovo, the necessary prerequisites for implementation of the adopted advanced technology are not yet available, since BOD_5 of the wastewater influent at the WWTP is only 41 *mg/l*. Similar cases of influent quality were found at most of the other WWTPs, where BOD_5 varied in the range from 40 to 120 *mg/l*. The remaining six WWTPs are designed adequately for the requirements of the relevant receiving waters, which are classified as "less sensitive" zones.

TABLE 1. Compliance of the WWTPs technological flowcharts with BOD and nutrients removal, and disinfection requirements

No	Municipal WWTPs at the towns of	Sensitivity Zones	Biological reactors operation mode	Envisaged nutrients removal		Disinfec tion agents
				N	P	
1	Gorna Oryahovitza	Sensitive	Denitr./Nitrific. + simult. coagulation	Yes	Yes	Cl_2
2	Montana	Sensitive	Denitr./Nitrific. + simult. coagulation	Yes	Yes	Cl_2
3	Lovetch	Sensitive	Extended aeration	No	No	Cl_2
4	Sevlievo	Sensitive	Extended aeration+ simult. coagulation	No	Yes	Cl_2
5	Popovo	Sensitive	Denitr./Nitrific. + simult. coagulation	Yes	Yes	Cl_2
6	Varna-Asparuh.	Primary treatment and pumping to the main WWTP of Varna				
7	Bourgas-Med. Rudnik	Sensitive	Extended aeration+ simult. coagulation	No	Yes	UV rays
8	Shumen	Sensitive	Conventional	No	No	Cl_2
9	Targovishte	Sensitive	Extended aeration	No	No	Cl_2
10	Pazardjik	Less sens.	Conventional	No	No	Cl_2
11	Blgoevgrad	Sensitive	Conventional	No	No	Cl_2
12	Razlog	Less sens.	Extended aeration	No	No	Cl_2
13	Smolyan	Sensitive	Denitr./Nitrific. + simult. coagulation	Yes	Yes	Cl_2
14	Madan	Less sens.	Extended aeration	No	No	Cl_2
15	Rudozem	Less sens.	Extended aeration	No	No	Cl_2
16	Zlatograd	Less sens.	Extended aeration	No	No	Cl_2
17	Dimitrovgrad	Sensitive	Extended aeration	No	No	Cl_2
18	Stara Zagora	Sensitive	Extended aeration	No	No	Cl_2
19	Haskovo	Sensitive	Denitr./Nitrific. + simult. coagulation	Yes	Yes	Cl_2

As mentioned above, the high rate of groundwater infiltration into the sewer networks is the reason for the highly diluted wastewater at the WWTPs inlets. Rehabilitation of the associated sewer networks is the most relevant solution of this problem. Such a radical measure, however, is very expensive and time consuming. Nevertheless this approach has been adopted by the government when establishing the National Programme for Sewerage.

In many cases, however, immediate technical measures are needed to be undertaken for coping with the urgent environmental problems. The problem with the wastewater dilution is not rare in the international sanitary engineering practice and the relevant technological approaches are available for overcoming this challenge.

The Sofia WWTP deserves special attention, because it is the largest plant in the country, with a current capacity of 480 000 m^3/d. It has been in operation since 1985 and now it is under reconstruction and upgrading, which is financed jointly by the PHARE programme and the concessionaire Sofiyska voda JSC in the amount of 21.6 million EUR. Reconstruction includes grit chambers, distribution chambers and aerated tanks, in addition to the sludge thickening and dewatering machines, which were replaced few years ago. Upgrading concerns secondary clarifiers, high rate digesters (methane-tanks), gas utilisation and electricity generation facilities.

Electricity produced by utilising methane gas is expected to cover about 60% of the electrical energy consumed at the WWTP. However, current reconstruction and upgrade do not consider the fact that the receiving water (the Iskar River, belonging to the Danube River basin) already belongs to the sensitive zone, as it became clear after the Ordinance for the Sensitive Zones came in effect in July 2003. Obviously the upgrade of the Sofia WWTP needs rearrangement and additional funding in the very near future.

As mentioned above, similar situations can be found almost in all of these 61 municipal WWTPs, which are currently in operation and belong to the sensitive zones (excluding the newly constructed WWTP at the town of Samokov in the Iskar River basin). Unfortunately, the same can be said about most of the WWTPs being designed and tendered (excluding the one in the City of Haskovo in the Mariza River basin and those in less sensitive zones). The following actions are urgently needed to be undertaken by the competent authorities to ensure wise spending of the resources already allocated and for the assessment of the need to secure additional funding:

- Revision of design projects and construction works of the WWTPs scheduled for construction, reconstruction and upgrading, if they belong to sensitive zones.
- Implementation of the best available technology (BAT) in the revision of WWTPs design projects.
- Revision of the financial chapter of the National Programme for WWTPs, where appropriate, to ensure compliance with the Ordinance for the Sensitive Zones.

6. General recommendations for BAT implementation

The best available technologies (BAT) suitable for the cases under consideration are the following:

- Denitrification/Nitrification (DN) for nitrogen removal
- Simultaneous coagulation (in the aerated tanks) for phosphorus removal
- Return sludge fermentation and lime precipitation for phosphorus removal (Phostrip and Modified Phostrip processes)
- Biologically enhanced phosphorus and nitrogen removal – BEPNR (UCT – University of Cape Town and Modified UCT processes)
- Technologies with separate sludge fermentor (eventually with solid wastes addition) as a part of technologies for biological phosphorus removal from diluted wastewater
- Sequencing batch reactors (SBR) application in the above technologies, where appropriate.

The technologies listed above are well known in the sanitary engineering practice and have proved their efficiency in a great number of municipal WWTPs all over the world. Their discussion is beyond the scope of this paper, since they have been described and discussed widely in the specialised literature. However, their applicability to the cases discussed above is of particular interest.

The DN technology for biological nitrogen removal is suitable in particular for upgrading the WWTPs with extended aeration indicated in Table 1, which belong to sensitive zones. Combination of the DN technology with simultaneous coagulation, Phostrip or Modified Phostrip processes for phosphorus removal, is also applicable where both N and P removals are necessary. In these cases the application of UCT and MUCT processes is also recommended, provided that the wastewater is not diluted. When wastewater is highly diluted (which is the usual case in Bulgaria), the application of technologies with separate sludge fermentor is strongly recommended.

Hydrodynamics of the relevant biological reactors is also very important with respect to saving volumes and expenditures. Consequently, the sequencing batch reactors (SBR) would be a good choice in many cases. In particular they are suitable for small and medium size WWTPs using biological processes for nutrient removal.

In the current world practice, the sludge liquor wasted from the processes of wastewater sludge treatment is recirculated to the WWTP influent. This creates significant additional loads of N and P to the main stream, thus increasing expenditures for its treatment. Separate sludge liquor treatment was recommended recently as a complement to the BAT already established.

New anaerobic technologies have been suggested recently and are currently investigated at the Technical University of Delft, the Netherlands, and in some other laboratories [7, 8]. Their application to nitrogen removal is very promising, in particular for sludge liquor treatment. These technologies became popular under the following terms [7]:

- Partial nitrification/Denitrification
- Sharon/Anammox
- Canon
- NO_x process.

Application of the above technologies for sludge liquor treatment will decrease the necessary bioreactor volumes and aeration capacity for the main stream treatment, thus leading to lower overall expenses for WWTP construction and maintenance. Our calculations show, for instance, that application of the Sharon/Anammox processes at a WWTP with capacity of 38 000 m^3/d leads to 25 % savings in comparison to the case with no sludge liquor treatment.

7. Summary and conclusions

About 3,500 million EUR have to be invested for the development, upgrading, reconstruction and rehabilitation of the urban water infrastructure in Bulgaria, including 2,200 million EUR for the sewer networks and WWTPs. Support from the international financial institutions and participation of the private sector are considered to be crucial importance for the implementation of relevant projects.

Currently 41 municipal wastewater treatment plants (out of 61 plants in operation in the country) use conventional secondary treatment technologies and serve 67 population centres with about 40% of the country population. In the Implementation Programme for WWTPs, 430 municipal WWTPs are scheduled for implementation by 2015, among which 61 are already in operation, 12 are to be completed in the near future and for 19 plants the design documentation is available. In Stage I of the National Programme for WWTPs (covering the period up to 2007) 36 municipal

WWTPs are included in the first priority group for construction, completion, reconstruction or upgrading at a cost of 325.5 million EUR.

Technological flowcharts of almost all the existing WWTPs, most of which are expected to require reconstruction and upgrading, do not comply with the requirements of the newly established Sensitive Zones Ordinance or meet them only partially. Furthermore, almost all of the WWTPs technological flowcharts include chlorination for disinfection, which is forbidden in the EU. In this respect disinfection with ultra-violet irradiation is recommended, which was originally planned only at one WWTP.

Significant infiltration is observed in almost all of the sewer networks currently in operation, which changes drastically the wastewater quality parameters and creates serious problems for implementing advanced wastewater treatment technologies, and biological nutrient removal, in particular. Nevertheless most of the WWTPs design projects do not consider the fact of wastewater dilution and assume "normal" wastewater quality, which makes them inadequate.

The general conclusion is that the flowcharts of the WWTPs under consideration need to be modified and additional funding be secured for their implementation in very near future. In this respect the implementation of several advanced wastewater treatment technologies is suggested, including those which are suitable for highly diluted wastewater.

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IMPROVING STP PERFORMANCE BY LIME ADDITION

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1. Introduction

Sewage treatment schemes typically include primary sedimentation designed to remove pollutant fraction associated with suspended matter, prior to the biological treatment. However, conventional primary treatment, which consists of plain sedimentation driven by gravitational forces, presents chemcial oxygen demand (COD) removal efficiencies in the range of 30-35%, in spite of the fact that the fraction of pollutants in the suspended phase may be higher than 70% of the total load. This leads to several drawbacks of the conventional sewage treatment schemes, including:

- high load to the biological section, with high energy demand for aeration
- high space required for the biological plants, due to the slow biodegradation of the suspended fraction of COD
- waste of energy that could be recovered through the anaerobic digestion of primary sludge.

To improve the overall sewage treatment scheme, a deeper understanding of sewage characteristics is required, not only in terms of total pollutant concentration, but also in terms of the size distribution of particles, which the pollutants are associated with. Pollutants in municipal sewage include a complex mixture of soluble and insoluble constituents with highly heterogeneous chemical composition and sizes. A large part of the pollutant load is in the insoluble form, ranging in size from less than 0.001 μm up to over 100 μm [1]. Pollutants are typically classified in four size ranges: settleable (> 100 μm), supracolloidal (1-100 μm), colloidal (0.001-1 μm), and soluble (< 0.001 μm). The limits of these operationally defined ranges may slightly change depending on the experimental technique used to separate the size fractions. Numerous works performed on size classification of organic pollutants in urban wastewater suggest that only a limited fraction of total COD in raw sewage may be considered truly soluble [2-7].

Biodegradability of the organic compounds present in sewage is strictly related to their size distribution. The majority of slowly biodegradable organic matter entering the biological section of a wastewater treatment plant can be assumed to be in the range of 10^3 amu to 100 μm. Whereas particles smaller than 10^3 amu can be directly taken up by cells, the utilisation of slowly biodegradable COD requires a preliminary

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slow step of hydrolysis [8]. Therefore, wastewater characterisation should include COD fractionation [9, 10].

The efficiency of primary treatment may be improved by aggregating finely dispersed matter into larger, easily settleable particles. Chemicals tested for sewage coagulation include iron or aluminium salts, lime, and polymers, which can be applied either by themselves or in combination with inorganic coagulants [11, 12]. Aluminium and iron salts may induce average removal efficiencies of about 91% and 73% for suspended solids (SS) and COD, respectively [13]. The main drawback of using iron or aluminium salts in raw sewage coagulation seems to be the significant increase in sludge production, which is well beyond that accounted for by the increased SS removal efficiency, being partly due to precipitated material [14]. High SS removal efficiency may be combined with low sludge production, by partly or fully substituting iron or aluminium salts with cationic polymers. However the use of synthetic, polymeric cations has been objected because of the possibility of toxic monomer formation.

The use of lime in sewage coagulation was mainly tested in coastal tourist areas, where high dosage lime treatment (pH 10.5-12) may result in significant disinfection (4-5 log reduction in *E. coli*), in addition to efficient SS and COD removals [12]. To prevent excessive sludge production due to the precipitation of calcium carbonate, a low dosage lime treatment (pH < 10) has been proposed, which may be still satisfactory in terms of SS and COD removal efficiency [16, 17].

This work was performed on actual sewage, withdrawn from the Roma-Nord Sewage Treatment Plant. With the aim of improving the conventional sewage treatment scheme, this work addressed three specific goals:

- to better characterise sewage in terms of COD partitioning among several size fractions
- to compare the biodegradability of the different size fractions, and
- to improve primary treatment with the addition of lime at low dosage (pH 9).

2. Materials and methods

2.1 WASTEWATER CHARACTERISATION

Roma-Nord STP is one of the four wastewater treatment plants serving the City of Rome. The plant (780,000 p.e., 354,000 m^3/d) consists of two parallel lines for sewage treatment, each including preliminary treatment (screening, sand and grease removal), primary settling, secondary treatment by activated sludge, and final disinfection by chlorination. In this study sewage samples were withdrawn from the effluent of the degritting tank, before primary treatment. In order to obtain size distribution of COD in degritted sewage influent, settling tests and filtration tests, employing sieves and membrane filters, were performed. Grab samples (20 L) of degritted sewage were split in three portions: the first small aliquot (10 ml) was used for COD measurements; the second one (about 2 L) was immediately used to measure the easily settleable fraction; and, the third portion (about 17 L) underwent sequential sieving and filtration. Micro and ultrafiltration tests were performed in a cross-flow filtration mode to minimise cake formation on the filter. The whole sequence included 3 sieves and 4 filters of the following decreasing pore sizes: 150-100-50-25-1-0.2 μm, and 100 kD (about 0.02 μm).

2.2 RESPIROMETRIC TESTS

In order to measure the amount of readily biodegradable COD (RBCOD) and slowly biodegradable COD (SBCOD) in raw and filtrate sewage, aerobic batch tests were performed under continuous stirring and at controlled temperature (25°C). For each test 1 L of biomass was withdrawn from Roma-Nord plant and aerated for 24 h in order to remove residual COD. Before starting the test, air bubbling was shut off and the biomass was allowed to settle for one hour. After removing 500 ml of supernatant, the concentrated biomass was kept aerated for 30 min, wherein endogenous oxygen uptake rate (OUR) was repeatedly measured. Then 500 ml of raw or filtered sewage was added at once, and OUR measurements were continued at regular intervals until the substrate was completely removed. To perform OUR measurements air bubbling was stopped and the rate of decrease of dissolved oxygen with time was monitored. To take into account oxygen transfer through the air-medium interface during OUR measurements, the transfer coefficient of the apparatus was measured in blank tests after inactivation of the biomass with HgCl (80 mg/l). In order to characterise the sludge under well defined conditions, batch tests with acetate as the only carbon source were initially performed.

2.3 PILOT TESTS ON SEWAGE COAGULATION WITH LIME

The coagulation-flocculation tests were performed on the degritted Roma-Nord sewage using two pilot plants. The first pilot plant was a continuous flow plant, which included two separate reactors for coagulation and flocculation, having a total volume of 170 and 520 litres, respectively. The coagulation tank was equipped with a marine-type propeller inducing shear rate values in the range of 314-795 s^{-1}. The flocculation tank was equipped with an anchor impeller, which provided mixing intensities in the range of 13-46 s^{-1}. The pilot plant did not include a settling tank. Settling was simulated through laboratory tests using an Imhoff cone.

Residence times of 10 minutes in the coagulation tank and 40 minutes in the flocculation tank were used. The unusually high residence time in the coagulation tank was prompted by the low dissolution rate of the lime slurry [15].

Lime was added (as a 2% slurry) at the inlet of the coagulation tank, using two peristaltic pumps that were controlled by a pH-stat system. After steady state conditions were established in the pilot plant (in terms of pH), pairs of samples of the influent and effluent wastewater were withdrawn as grab samples from the feed of the coagulation tank and from the outlet of the flocculation tank, respectively. Sampling of the effluent wastewater was delayed, with respect to the corresponding influent sampling, by a time equal to the total residence time in the pilot plant. A portion of the grab samples was used for pH and COD measurements, whereas the remaining portion was immediately used for a settling test in an Imhoff cone. After one hour settling, the residual COD in the supernatant liquor as well as the volume and the concentration of the settleable solids were measured.

The semi-batch plant was a cylindrical reactor (1.6 m in internal diameter), with a conical bottom for sludge settling. The reactor was equipped with an anchor impeller. A coagulation-flocculation test included the following sequential steps:

1. Filling the tank with degritted sewage: during this phase mixing was on and pH was maintained at 9 with the addition of 2% lime slurry using a peristaltic pump controlled by a pH-stat system.
2. Flocculation: after filling the reactor up to a pre-set level, feed pump and lime pump were stopped, whereas stirring was kept on for 40 additional minutes.
3. Settling: settling was simulated by Imhoff settling tests on samples withdrawn from the reactor immediately after stopping the stirrer, in order to do an unbiased comparison with the results obtained in the continuous flow pilot plant.
4. Discharge: at the end of the test the reactor was completely emptied through the sludge outlet at the end of the conical bottom.

2.4 ANALYTICAL METHODS

COD was measured using the Spectroquant COD cell test supplied by Merck (digestion with commercially available reagents followed by colorimetric measurement of excess dichromate). Replicate measurements of two potassium phthalate samples at COD levels of 160 and 275 mg/l showed coefficients of variation of 1.7% and 1.4% respectively, and accuracy within 10%. In the highly heterogeneous samples of raw sewage, higher coefficients of variation were found (around 8%), likely due to the use of small sample aliquots (2 ml). To offset the lower precision of the method with commercial vials in samples containing suspended solids, in this work all the COD measurements were performed in three replicates.

Oxygen concentration measurements for OUR determination were performed using the WTW oxygen probe.

Quantitative *E. coli* data (MPN) on raw and settled sewage were gathered using the QuantiTray-Colilert system, a commercially available enzyme substrate test, which conforms to Standard Methods [18].

3. Results and discussion

3.1 COD FRACTIONATION

A half-year long survey of the degritted Roma-Nord sewage was devoted to the classification of influent COD into the four conventional size ranges of settleable, supracolloidal, colloidal and soluble organic matter. Before that, a preliminary study was performed on the variability of sewage characteristics during the day. As expected, it was observed that sewage strength, in terms of COD, may more than double at the peak hours, compared to the minimum value. In addition, the hourly variability in sewage strength is accompanied by a significant change in the size distribution of COD. Thereafter, sewage sampling was performed at noon to offset the hourly variability of sewage characteristics.

In Table 1 the results of the survey are compared with literature data. Because the operationally defined size intervals are not standardised, they are also reported (in parentheses) in the table. The settleable fraction determined in this work is in good agreement with data typically obtained in urban wastewater. The supracolloidal fraction yields a contribution comparable to that of settleable COD. In contrast, the contribution of the colloidal fraction seems to be lower than those typically reported in the literature. This can be ascribed to the fact that in this survey colloidal and soluble fractions were separated via filtration through 0.2 μm, with the consequence that the so defined "soluble" fraction may include part of the colloidal COD. Similarly, the very high "soluble" fraction determined in Rickert and Hunter's work [4] is probably due to the fact that these authors use centrifugation for physical separation between supracolloidal, colloidal, and soluble fractions. Centrifugation might have been less efficient than filtration for separation of the three fractions according to the nominal size intervals indicated.

TABLE 1. COD distribution in domestic wastewater

Reference	Total COD (mg/l)	% of COD in size fractions			
		Settleable	Supracolloidal	Colloidal	Soluble
[4]	418	29	21 (settleable – 1 μm)	10 (0.001 – 1 μm)	40 (< 0.001 μm)
[5]	498	43 (> 106 μm)	30 (106 – 3 μm)	15 (3 – 0.025 μm)	12 (< 0.025 μm)
[9]	410	27	39 (settleable – 1.2 μm)	34 (< 1.2 μm)	
[6]	430	29	31 (settleable – 1.2 μm)	31 (1.2 μm – 10 kD)	9 (< 10 kD)
This work	162-392	34-49	30-46 (settleable – 1 μm)	3-10 (1-0.2 μm)	13-28 (< 0.2 μm)

To better understand the distribution of particulate COD in the influent sewage, the complete sequential procedure of screening, microfiltration, and ultrafiltration was used. The histogram in Figure 1 shows a typical COD distribution among the different size fractions. A very large fraction of COD is associated with particles larger than 150 μm. Because the easily settleable fraction of the sample under study was about 49%, Figure 1 suggests that all the particles larger than 50 μm are included in the settleable fraction, in spite of the fact that settleable COD is conventionally associated with particles larger than 100 μm.

Among the supracolloidal fractions, particles in the size range 25-50 μm seem to be predominant. The colloidal and soluble fractions (< 1 μm) add up to about 31% of the total COD. In this case a better separation between colloidal and soluble fractions suggests that the colloidal fraction predominates over the soluble one, which is only about 11% of total COD. This is in agreement with the results of Vaillant *et al.* [6], who used similar ultrafiltration techniques to separate soluble from colloidal COD.

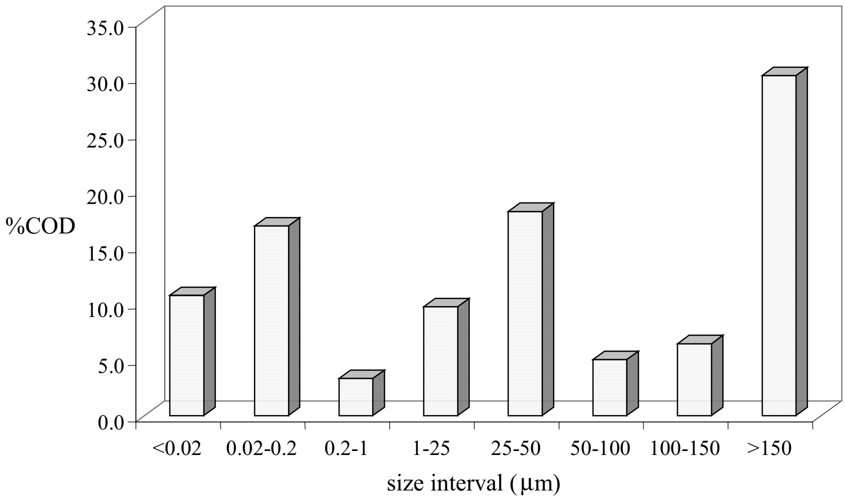

Figure 1. Distribution of COD among size fractions

3.2 BIODEGRADABILITY

In order to characterise the biodegradability of different COD fractions, the Oxygen Uptake Rate (OUR) method was used. As reported in the literature, preliminary tests with acetate as the sole carbon source are necessary. Three replicate tests with acetate gave a mean yield value (biomass produced, in terms of COD, per unit amount of COD consumed) of 0.78 COD/COD, which is typical for respirometric tests performed using urban activated sludge spiked with acetate [19].

Respirometric tests with raw sewage provided the typical OUR profile reported in Figure 2. The OUR profile can be easily divided into a high-OUR phase (phase I, corresponding to readily biodegradable COD depletion) and a low OUR phase (phase II, related to the consumption of slowly biodegradable COD). Using the yield value obtained with acetate tests, COD consumed in phase I and in phase II may be easily calculated. Preliminary results with raw and filtered sewage suggest that only about 36% of COD is readily biodegradable, and that size fractions below 0.2 μm do not contain slowly biodegradable COD. These results are in satisfactory agreement with the work of Levine *et al.* [20], who found that for colloidal particles the rate constants for biodegradation were significantly higher than those calculated for supracolloidal particles.

The results of the biodegradability studies suggest that eliminating the slowly degradable particles prior to aerobic biological treatment promotes more effective utilisation of the biological treatment capacity.

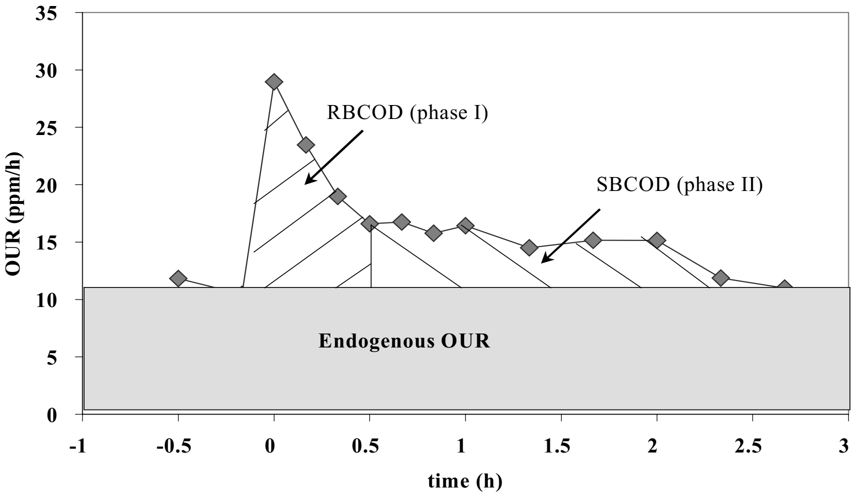

Figure 2. Typical OUR profiles in batch test with raw sewage

3.3 PILOT PLANT TESTS

3.3.1 *Suspendend COD removal*

Table 1 shows that over 30% of COD in Roma-Nord influent sewage is not easily settleable, even though it is still in suspended form. In addition, the biodegradability studies indicated that this large portion of COD entering the biological section is not easily biodegradable, requiring a preliminary hydrolysis step. These results suggest that the overall treatment system could be improved by aggregating the finely dispersed COD in order to remove it in the primary section. To investigate the potential removal of finely dispersed COD in primary treatment, lime coagulation tests were carried out in the continuous flow pilot plant.

Previous process optimisation studies suggested working at pH 9 to avoid excessive sludge formation due to precipitation of calcium carbonate [21]. Therefore, in this work sewage coagulation tests at pH 9 were compared with plain settling of the Roma-Nord influent sewage. Figure 3 compares treatment efficiencies, in terms of percentage of COD removed, obtained in Imhoff settling tests on the degritted sewage and on the effluent from the continuous flow pilot plant. Plain settling may remove influent COD with an average efficiency of about 30%, in agreement with average efficiencies typically reported for the conventional primary treatment. In contrast, the lime-enhanced primary treatment shows an average treatment efficiency of about 65%. This additional 30-35% efficiency of COD removal is consistent with the hypothesis that coagulation may induce an almost complete removal of the supracolloidal fraction in primary treatment.

Within the experimental conditions tested in this work (mixing intensities: 314-795 s^{-1} for coagulation and 13-46 s^{-1} for flocculation), mixing conditions did not significantly affect the lime-enhanced process, which seems to be controlled by the slow lime dissolution step.

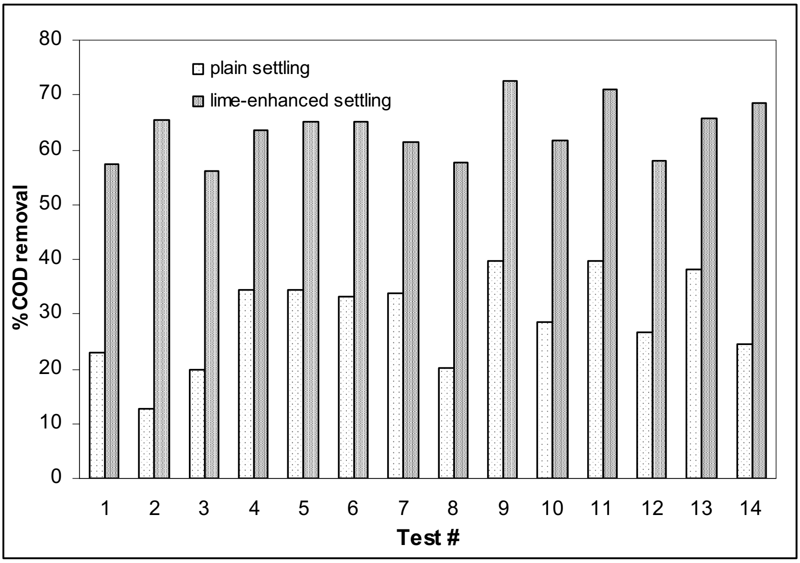

Figure 3. Comparison between plain and lime-enhanced (continuous flow pilot plant) settling of suspended COD from Roma-Nord sewage

3.3.2 *Lime consumption and sludge production*

Lime dosages in the range of 152-327 mg/l were required to keep the pre-set pH 9 in the coagulation tank, whereas the amount of lime required in sewage titration to increase pH up to 9 was in the range of 30-80 mg/l, with an average of 57 mg/l. The highly variable lime dosages observed both in pilot plant and in titration tests at pH 9 suggest that, in spite of the relatively constant pH, buffer capacity of influent raw sewage varies significantly and it is not correlated with sewage strength measured in terms of such gross parameters as COD and SS.

To try to explain the unexpectedly high lime dosages required in the pilot plant tests at pH 9 at least two hypotheses should be explored. Firstly, it must be emphasised that lime dissolution is a relatively slow process, implying transfer of matter through the solid/liquid interface [22]. In addition, the dissolution rate of commercial lime is not easily predictable, varying significantly with lime source, particle size, and slurry concentration [23]. Particle size seems to be the most important factor affecting lime dissolution rate [24]. In light of these considerations, it is likely that part of the added lime remains undissolved during the coagulation process, even though an unusually high residence time (10 minutes) was used in the coagulation tank of the continuous flow pilot plant. The second consideration is related to the potential precipitation of calcium carbonate, a side reaction which consumes lime. A thermodynamic model of the chemical system representing inorganic components of the raw sewage under study predicts calcium carbonate precipitation even at pH as low as 9 [16]. Even though formation of thermodynamically stable dolomite and calcite is unlikely in the experimental conditions used in the pilot plant tests, precipitation of amorphous calcium carbonate is still possible, mainly in localised supersaturation conditions.

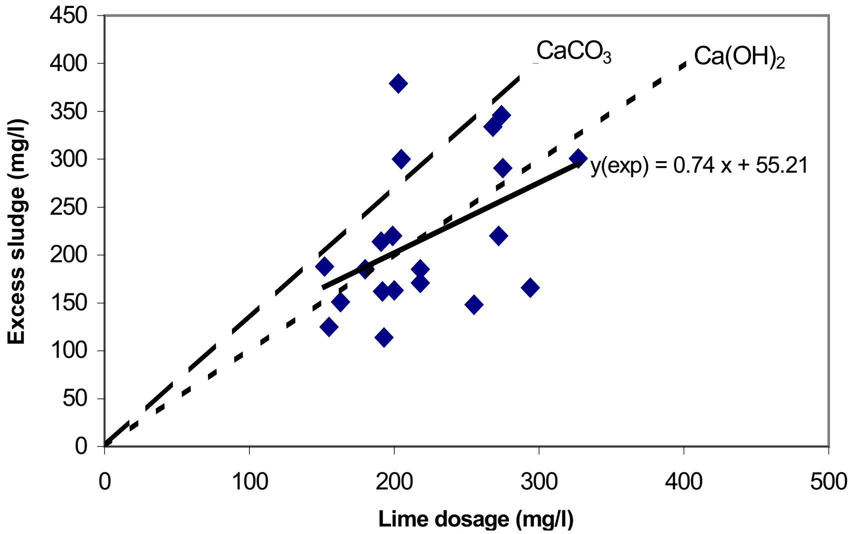

Figure 4. Excess sludge produced at pH 9 as a function of lime dosage

Both hypotheses described above imply that in pilot plant tests some extra sludge must be formed in addition to the amount of sludge representing the fraction of suspended solids removed from degritted sewage. The excess sludge may be calculated as the difference between the amount of sludge produced in the pilot plant test and the amount of SS removed from the influent sewage. Excess sludge values are reported in Figure 4 as a function of lime consumption in the pilot plant tests at pH 9. The two dashed straight-lines displayed in Figure 4 show the expected relationships based on two hypotheses: 1) incomplete dissolution of lime and/or 2) calcium carbonate precipitation. The two theoretical relationships use the very simplifying assumption of neglecting the amount of lime required to increase sewage pH up to 9. On the basis of this simplifying assumption, we would expect a straight-line with slope of 1 for the hypothesis of negligible dissolution of lime. Alternatively, a straight-line with slope of 1.35 should be expected for the hypothesis of calcium carbonate precipitation. Figure 4 shows highly dispersed experimental results. The solid straight-line interpolating the experimental results has a very low regression coefficient (R^2 = 0.21). The 95% confidence interval of the slope is very large (0.036-1.43), including both the expected slope for negligible dissolution of lime and the expected slope for calcium carbonate precipitation. Due to the large variability of the pilot plant results, the relationship between excess sludge production and lime consumption is not sufficient to infer, with a high degree of confidence, the composition of the excess, inorganic sludge produced at pH 9. At a lower degree of confidence, the 90% confidence interval of the slope is still rather large (0.16-1.31), but it suggests that the hypothesis of incomplete dissolution of lime is more consistent with the experimental results.

The hypothesis that excess lime consumption and consequently excess sludge production are due to insufficient lime dissolution in the coagulation tank of the continuous flow pilot plant suggested to carry out parallel experiments on sewage coagulation with lime in a semi-batch plant, following the procedure described in the experimental section. In this case the long residence time during the first step (sewage feeding and lime addition) that lasted 35 minutes was expected to be more favourable for the complete dissolution of lime. Table 2 shows that, on the average, the continuous flow plant consumes 47% of additional lime to coagulate Roma-Nord sewage at pH 9. This supports the hypothesis that the excessive amount of lime consumed and the consequent excess sludge produced in the continuous flow tests are due to incomplete lime dissolution.

TABLE 2. Lime consumption in coagulation tests at pH 9 carried out in continuous flow and semi-batch pilot plants

Raw sewage characteristics		Lime dosage (mg/L)	
pH	COD (mg/L)	Continuos flow	Semi-batch
7.78	226	154	
7.82	227	154	
7.88	235	156	
7.72	238		109
7.78	244		106
7.88	203		94
7.91	279		97
7.66	226	142	
7.71	202	140	

3.3.3 *Pathogens removal*

To determine the sanitary quality of water samples, tests for detection and enumeration of indicator organisms, rather than pathogens, are used. In this regard, experience suggests that coliform group density is a significant criterion of the degree of microbiological pollution. In this work *Escherichia coli* was chosen as an indicator of faecal contamination. The estimation of *E. coli* density both in raw and treated sewage was carried out using the commercial enzymatic Quanti-Tray test as described in the experimental section.

Table 3 compares *E. coli* counts in plain and lime enhanced settling. The *Escherichia coli* concentration in the raw Roma-Nord sewage is in the typical range of values reported in the literature for domestic wastewater (10^6-10^7 #/100 ml). In accordance with the literature data, plain settling, as used in conventional primary treatment, shows little or no effect on *E. Coli* counts in the sewage (being still in the range of 10^6-10^7 #/100 ml). In contrast, Table 3 shows that the lime-enhanced settling may decrease bacterial counts by one or two orders of magnitude. Even though not as large as the one reported for the lime treatment at pH 10.5-12, this disinfection ability may be an additional benefit in favour of the lime-enhanced primary treatment.

TABLE 3. *E. coli* counts (#/100 ml) in raw and settled Roma-Nord sewage

Raw sewage	Settled sewage	
	Plain settling	Lime-enhanced settling
1.3×10^7	1.4×10^6	6.2×10^4
1.5×10^7	1.7×10^6	1.2×10^5
1.2×10^7	1.1×10^7	2.3×10^5
1.7×10^7		2.3×10^6

4. Conclusions

- COD in Roma-Nord sewage is predominantly associated with settleable and supracolloidal particles, each size class containing about 40% of total COD.
- A large fraction of COD associated with supracolloidal particles is characterised by slow degradability, therefore suggesting that removal of these particles prior to biological treatment may greatly improve the overall treatment scheme.
- Pilot plant coagulation tests with lime at pH 9 showed that the lime-enhanced primary treatment may increase COD removal efficiencies from typical 30-35% up to 65-70%, by inducing an almost complete removal of the supracolloidal COD fraction.
- An additional benefit of lime-enhanced primary treatment may be a certain degree of disinfection (1-2 log reduction in *E. coli*), which is not typically obtained in conventional primary treatment.
- The main drawback of sewage coagulation with lime is the consumption of a large amount of lime and the consequent production of excess sludge, most likely due to the incomplete dissolution of lime. This drawback may be overcome by using a semi-batch coagulation process in place of the continuous flow mode.

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RESTORING NATURAL WATERWAYS IN DENVER, USA AREA

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1. Introduction

The State of Colorado Legislature in 1969 established the Urban Drainage and Flood Control District (District), primarily in response to the disastrous 1965 flood on the South Platte River. Its main mission is to assist cities and counties in the Denver metropolitan area with drainage and flood control problems. The District covers an area of 4,116 km^2 and has approximately 2,600 km waterways that drain 350 ha, or more of tributary area. The present population of the District is approximately 2.3 million people.

For over 30 years, the District has observed that the process of urbanisation profoundly changes the surface runoff hydrology in this semi-arid region of the United States. With an annual average precipitation of 360 mm, the number of observed surface runoff events increase from less than one to over 29 on an average annual basis. In addition, many streams that were ephemeral develop baseflows as a result of outdoor water uses by its residents. In response to these changes, the natural waterways erode downward and experience accelerated bank erosion as they geomorphically adjust to reduce their longitudinal slopes (see Figure 1).

The District's goal for many years has been, as much as financial resources permit, to prevent such degradation from occurring and to rehabilitate degraded waterways. Over the years the emphasis has shifted from pure engineered solutions to stabilise urban waterways to solutions and techniques that address aesthetic and ecological improvements of the waterways system. This shift in the way they are stabilised was found to have only a small incremental cost increase over projects that do not address aesthetics and ecological issues. However, the payback for these increased costs include much improved community relations, significant multi-functional benefits to the community (i.e., enhanced recreation, wildlife and aquatic habitat potential) and no loss in the long-term economic life and stability of the rehabilitated waterways.

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Figure 1. Example of an eroded waterway in response to upstream urbanisation
Photograph by permission of the Urban Drainage and Flood Control District.

The overall District's waterway rehabilitation and protection program's goals can be summarised as follows:

- Maintain and restore flood carrying capacity of the waterways
- Protect private and public property and infrastructure from flood damages
- Arrest vertical degradation and excessive lateral migration using techniques that blend into the waterway's natural setting, provide an asset to the community and enhance its ecology
- Acquiring right-of-way to preserve the 100-year floodplains whenever affordable.

What follows are several examples of waterway rehabilitation efforts. Some were major capital projects, while others were accomplished as a part of the District's annual restorative and rehabilitative maintenance activities.

2. Example # 1. Willow Creek Channel Rehabilitation and Sedimentation Trap

Willow Creek is a waterway located on the south side of the Denver metropolitan area. In response to many square kilometres of newly urbanised lands, it carved a 9-metre high vertical bank that undercut residential yards and endangered private residences (see Figure 2). The project sponsors were willing to use bioengineering techniques rather than traditional exposed rock armouring methods. In order to stabilise the 9-metre tall bank, buried riprap was used to reinforce and protect the toe of the slope. The lower portions of the slope itself were backfilled and stabilised using soil wrap lifts. Above these lifts, brush layering along the face of the slope was employed to slow the surface runoff and to trap organic material.

Over the last 15- to 20-years a healthy stand of trees developed on the side of the creek opposite the eroded vertical bank. To rebuild the vertical bank at a slope of 2H:1V, the low flow channel of the creek had to be relocated 15 metres to the other side of the stand of trees (see Figure 3). Again, buried riprap and fibre rolls were used to strengthen the new toe of low flow channel's bank, where willows were staked along the water's edge.

Figure 2. Willow Creek's eroded bank threatens homes.
Photographs by permission of Urban Drainage and Flood Control District

Figure 3. Willow Creek relocated from to the far side of the trees.
Photographs by permission of Urban Drainage and Flood Control District

The longitudinal grade of the stream was stabilised using two grouted boulder grade control structures, one immediately upstream and one downstream of the rehabilitated channel reach.

Immediately downstream from the rehabilitated reach of Willow Creek, sediment was being deposited into a large stormwater detention basin (Englewood Detention). This was repeatedly burying wildlife habitat and a pedestrian trail. To help reduce this ever-progressing sediment aggradation in the detention basin and loss of flood storage, wildlife habitat and function of a trail, a sediment trap was constructed (i.e., forebay) upstream of Englewood Detention basin. The tributary watershed has over 20-square kilometres and it was not possible to retrofit a sediment trap that would meet the

District's latest water quality criteria [1]. A smaller sediment trap that has a volume of 765 m^3 was installed instead. Since its location was highly visible to trail and open space users, the design included a natural rock spillway, landscaping, and many plantings to give it an appearance of a pond in a natural setting (see Figure 4). Since its construction in 1998, approximately 2,800 cubic metres of sand are removed each year from this facility, significantly reducing the rate of loss of the downstream wildlife habitat and stormwater detention (i.e., flood storage) volume.

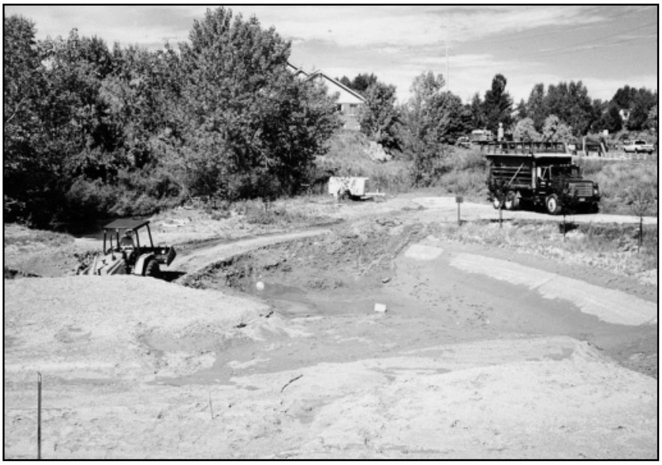

Figure 4. Sedimentation pond during a cleaning operation.
Photograph by permission of Urban Drainage and Flood Control District

3. Example # 2. Goldsmith Gulch Channel Rehabilitation in Bible Park

Goldsmith Gulch flows through the middle of a 30-hectare Bible Park that is located within the south end of the City of Denver. Upstream of this park there is a 15.5 square kilometre watershed that changed in 25 years from mostly rangelands to densely developed urban areas. In response to this urbanisation, Goldsmith Gulch eroded downward to form a 1.5 to 3.0 metre deep incised low-flow channel with vertical banks (see Figure 5). Since the creek within the park had a wide undeveloped floodplain, an opportunity existed to create a meandering riparian corridor and to enhance many of the park-related aesthetic and wildlife habitat features.

After considerable analysis and public input, it was decided to reduce the longitudinal grade of this 790-metre reach of the creek from 0.6 to about 0.2 percent. This required that the 3.2 metres of elevation change in the flow line be flattened through the use of two grouted boulder drop structures. These drop structures divided the creek within the park into three distinctly different channel reaches.

In the lower reach, boulder walls along the low-flow channel were used to create an island that protected a large area of trees adjacent to the channel. It also provided a more interesting and diverse conditions for park users to experience. The reduced velocities and constant inundation in the widened low flow channel upstream and downstream of the island resulted in flourishing wetland areas.

Figure 5. Degraded Goldsmith Gulch through Bible Park, note exposed utility line.
Photograph by permission of Urban Drainage and Flood Control District

The middle reach was changed form a linear, deeply incised, alignment to a meandering shallow stream. The result was a broad, gently sloping, riparian corridor that is inundated several times a year, thereby sustaining wetland vegetation and adjacent riparian trees and brush (see Figure 6a and 6b). The low-flow channel banks for this reach were protected with a soil-riprap mixture that was vegetated with wetland species and willow plantings.

Figure 6a. Goldsmith Gulch immediately after rehabilitation.
Photograph by permission of Urban Drainage and Flood Control District

Figure 6b. Goldsmith Gulch four years after rehabilitation.
Photograph by permission of Urban Drainage and Flood Control District

The upper reach was not as severely incised as were the downstream reaches and most of the existing channel alignment was maintained. To arrest further downward degradation of this reach that would undermine the existing banks and damage mature trees growing along its banks, a 1.1-metre high drop structure was installed at its downstream terminus.

4. Example # 3. South Platte River Stabilisation

The South Platte River Program was established in 1987 with the goal to address the unique challenges associated with the largest multi-jurisdictional waterway located in the centre of an urban area. This program has an annual maintenance budget that provides funds to address ongoing system deterioration problems and to control erosion of the river's channel. The maintenance program's objectives are to:

- Maintain and restore flood carrying capacity to protect properties from flooding
- Arrest the river's vertical degradation and excessive lateral migration
- Acquire right-of-way for access and flowage capacity preservation

Maintenance activities that repair or rehabilitate degraded and damaged elements along the South Platte River in the Denver metropolitan area have evolved over a 15-year period into an environmentally sensitive approach as compared to river maintenance practices used in the past for flood-control purposes only. One of the lessons learned early in this program was that the use of the so-called bio-engineered or "soft" techniques that used only vegetative materials such as willow bundles, stakes and grasses has consistently resulted in bank failures. As a result, the use of combined "hard" and "soft" techniques was found to provide long-lasting stabilisation and rehabilitation improvements while also providing very diverse riparian corridor and wildlife habitat along the river. The overall rehabilitative maintenance program for the river is based on the following:

- Monitor the river closely for excessive river degradation and riverbank erosion/migration.
- Install grade control structures to check degrading reaches that provide for fish migration, aquatic habitat, and boater safety.
- Combine "hard" and "soft" bank stabilisation treatments to help avoid immediate and ongoing bank failures that result from high flow velocities and sustained flows.
- Aggressively control invasive species of non-native vegetation and replace them with native species whenever opportunities arise.
- Employ the services of ecologists and landscape architects to help design projects.
- Pursue acquisition of floodplain properties to preserve flow capacity and floodplain storage.

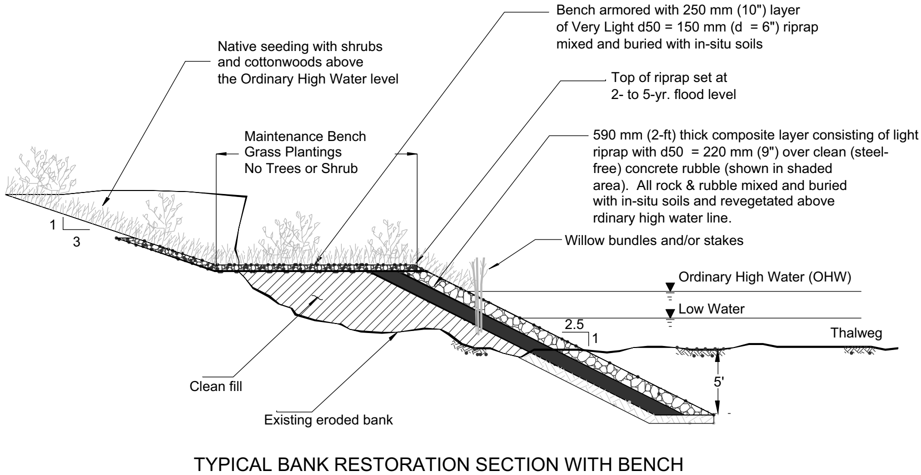

Figure 7. Typical cross-section for riverbank rehabilitation along South Platte River.
Figure used by permission of Urban Drainage and Flood Control District

These recommendations have resulted in many successful restorative maintenance projects for the District's South Platte River Program. One of these was the left-hand bank of the river in south Adams County (i.e., north of Denver). It was badly eroded in the summer of 1998 by a thunderstorm-produced runoff event that peaked at 370 m^3/s. This undermined a recreational trail and endangered adjacent property. This bank damage was rehabilitated to a condition that was an improvement over the bank conditions (1H:1V) that existed prior to the erosion damages. A total of 460-metres of riverbank was laid back at a slope of 2.5H:V and flatter and 2,600 m^3 of rock riprap having d_{50} = 230 mm, was buried into the river's bottom at the toe of the slope and for 1.2 metres above the low flow water surface (see Figure 7). All riprap was mixed with local soils to fill the voids, covered by 150 mm of topsoil and revegetated with native grasses. Locally harvested cottonwood and sandbar willow cuttings were staked along the lower bank. Two years after construction the vegetation was maturing and the bank was showing that the rehabilitation efforts were succeeding (see Figure 8).

Another river rehabilitation project restored some of the river's grade at a historic flow-measuring gauge in Henderson, Colorado. The river's bottom over the last 40 years dropped in elevation approximately 1.6 metres and the gauge was not able to measure low flows in the river. Since the river's bottom was degrading and causing bank failures upstream, it was decided to install a 1.2-metre high grouted boulder drop structure. Its purpose was to restore, at least in part, the bottom elevation of the river at this site while providing: (1) a safe passage for boaters, (2) little impediment to fish migration, (3) reaeration of the low flows to improve the dissolved oxygen levels during warm months of the year, and (4) a blending of this grade control structure into the adjacent terrain as much as possible.

Figure 8. An eroded South Platte River bank in Adams County in 1998 and how the rehabilitated bank looked in 2001.
Photographs by permission of Urban Drainage and Flood Control District

Over the years much concrete rubble and other trash was dumped on the river's bottom and its banks. All of this trash was removed, the site regraded and then revegetated with native vegetation and grasses. Unfortunately the very dry weather that occurred in 2002 retarded the reestablishment of the new vegetation (see Figure 9), but it has greened up more when precipitation patterns returned to near normal in 2003.

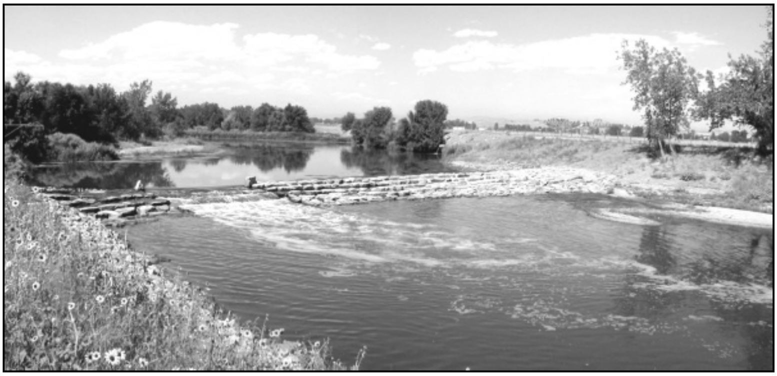

Figure 9. A 1.2 m high grouted boulder grade control structure at the Henderson flow measuring gauge site; safe for boaters and permits fish migration.
Photographs by permission of Urban Drainage and Flood Control District

5. Boater Safety

Along the South Platte River and some of its tributaries rafting, canoeing, kayaking, and other water-based recreational activities occur. Although the ultimate responsibility for prudent use of urban waterways resides with the individual, the design of hydraulic structures in these waterways requires an added standard of care that address safety concerns for the public. Even very low vertical or sloping drops can create "keepers" that trap upended boaters against the downstream face of the structure. There have been several drownings in the Denver area attributed to these types of structures (see Figure 10).

In response to public concerns, in the late 1970s the District started to retrofit some of these so-called "killer drops" with boat chutes. The key elements in their design included a much flatter longitudinal slope, sometimes done in a series of steps, and a geometry that develops a focused flow that prevents a lateral hydraulic jump from forming. Instead, an undulating central jet is created that does not have reverse surface flows and moves the boaters through the bottom of the drop without trapping them. An example of one of the early retrofit project is the 3rd Avenue drop structure. Originally, it was a 2-metre drop with a smooth sloping face, similar to an Ogee weir in cross-section. In the late 1970's two rafters were trapped and drowned at this site before rescue teams could get to them. The dam was cut and a boat chute having the same crest elevation was extended upstream to create the needed geometry (see Figure 11). A side benefit that was discovered afterwards was that some of the fish species that migrate up and down the river were now able to migrate up the chute because it had areas along its profile for them to get out of direct currents and rest.

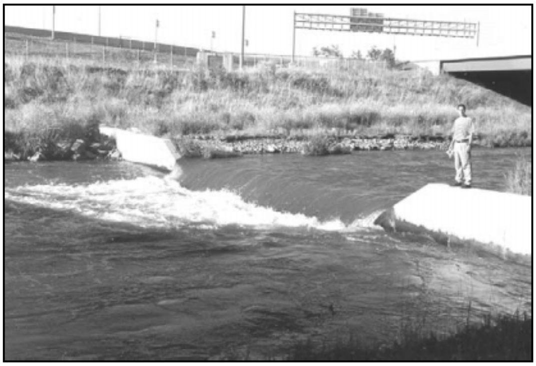

Figure 10. A 0.8-metre high "killer drop" in Clear Creek; a site of two drownings. "Keeper" at the downstream face has currents that pull objects against the drop.
Photograph by permission of Wright Water Engineers, Inc.

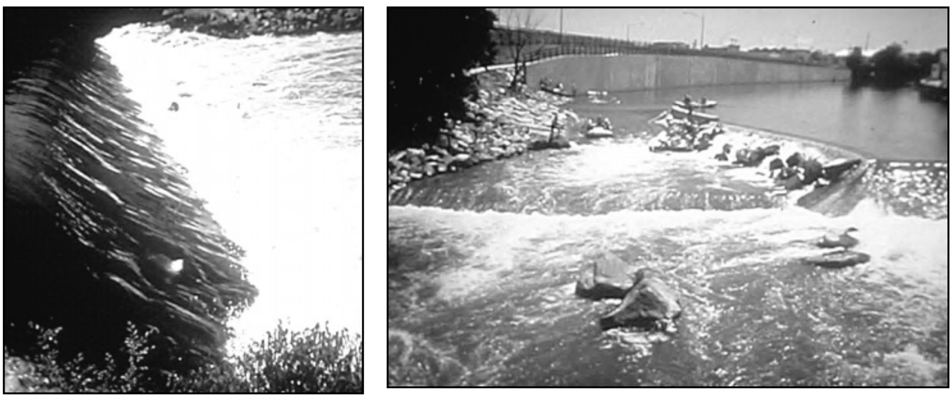

Figure 11. A 2-meter high "killer drop" modified to include a boat chute. Note the strong "keeper" in "before" and the strong central "jetting" current "after" pictures.
Photograph by permission of Urban Drainage and Flood Control District.

6. Lessons Learned

The Urban Drainage and Flood Control District has learned many lessons since its creation in 1969. The one overriding theme is that as lands urbanise, the receiving waterways (i.e., gulches, streams and rivers) experience observable changes in their geometry and biota. There is a need to both protect these receiving waterways and to rehabilitate them. Doing this under a government regulatory process can be costly, slow and cumbersome. Nevertheless, it is something the District has come to believe to be a very important part of maintaining quality of life and protecting the water environment in urban centres. As a result, the District has updated its Criteria (2) to address this issue and now suggests the following set of recommendations be followed when doing stream and river rehabilitation projects. They are based on years of in-the-field observations of successes and failures, are pragmatic, and provide reasonable safety to the public:

1. Involve the public in the conceptualisation process of projects and programs.
2. Address the governmental regulatory requirements.
3. Employ the services of ecologists and landscape architects to help design projects.
4. Monitor waterways for excessive degradation and bank erosion.
5. Address observed degradation problems as soon as possible after they are first observed.
6. Install grade control structures to check downward incision of the waterways as watersheds begin urbanising to reasonable levels, say no more than one metre. Installation of grade control structures before problems develop is the least costly means of stabilising streams impacted by urbanisation.
7. Install drop structures to rehabilitate degraded streams and rivers to check further downward incision of these and to restore, at least in part, some of the degraded stream elevation. In doing so, provide for:
    a. Fish migration and aquatic habitat,
    b. Boater safety (no keepers), and
    c. Improved aesthetics by blending into the surrounding landscape.
8. Use combination of "hard" and "soft" techniques to provide for long-term stability and life of a project and natural vegetation along banks of streams and rivers.
9. Aggressively control invasive species of non-native vegetation and replace with native species whenever opportunities arise.
10. Acquire floodplain properties whenever funds and opportunities permit. These ownerships allow better maintenance, building of publicly owned parks and trails, and preserve open spaces for wildlife habitat and public to enjoy.

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URBAN STREAM RESTORATION STRUCTURES

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1. Introduction

The growth of urban centres had led to a decline in the quality of numerous watercourses and riverine ecosystems. Different form of degradation has occurred in water course not only because of impacts of urbanisation on the hydrologic and hydraulic conditions, but also because of the presence of in-stream works. Concrete and other hard structures have modified the natural corridor characteristics, causing its progressive degradation. Consequently the necessity of mitigating some of these works has brought a new attention to stream restoration, which is defined as the structural and functional return of a degraded riverine ecosystem to its pre-disturbance conditions.

Figure 1. Sill grade control (left) and wall with concrete toe protection (right) in a mountain urban stream

2. The effects of urbanisation

An urban stream can be identified as a watercourse which crosses cities and villages, both in a mountain and in a floodplain area. Urbanisation directly impacts stream and riparian corridors because it modifies channel geometry through straightening, lining, or placement in culverts, it reduces riparian corridor width, determines the increase of pollutant loading and changes the sediment yield [1]. Typically high sediment loads with finer particle sizes are produced during urbanisation, while sediment transport diminishes after the process of development is completed. Urbanisation changes the runoff and sediment yield characteristics, which are the principal

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variables that define the shape, character, and quality of a watershed's streams: peak storm flows are greater and more frequent, the duration of stream flows capable of altering channel beds and banks becomes longer, the channel tends to enlarge through incision and widening processes, and stream temperatures and pollutant loadings increase. In addition to increasing runoff, urbanisation decreases the magnitude of baseflows by limiting infiltration, and increases the duration and frequency of runoff events.

Figure 2. Channelised and confined stream in a heavily urbanised area (left) and sewer outfalls (right)

In the urban environment, streams are often channelised with regular banks (generally confined between walls or levees) and concrete revetments, which are necessary to increase the conveyance of flood flows and maintain a stable bed. The use of concrete and hard structures adversely affects biological diversity in streams. The abundance and diversity of fish and aquatic insects decreases markedly. In heavily urbanised streams, ambient riparian areas are generally absent and the stream can not expand laterally. Native species are often displaced by non-native species having higher pollution tolerance. The pollutants deposited from the atmosphere during storms are quickly washed off and rapidly delivered to aquatic systems, where they heavily damage the biota of the stream and reduce its potential use for recreation.

Generally it is possible to make a distinction between mountain urban streams and floodplain ones. Mountain urban streams (*Figure 1*) are characterised by incised sections and are often confined between walls. Bed material is constituted by cobbles, pebbles and sometime boulders. Problems of erosion are common and grade control is necessary to avoid the failure of the near stream infrastructure. The flood events are sudden and violent and in such conditions shear stresses are very high.

In floodplain areas streams are generally channelised and straight. Sewer and culvert inflows are frequent (*Figure 2*). Bed material is sand or cohesive material. Levees and natural banks are present in less urbanised areas, while confining walls are frequent in densely urbanised areas. Concrete revetments are intensely used to protect banks from erosion (*Figure 3*). Flood events are generally less sudden and violent that in mountain watercourses.

Figure 3. Concrete bank protection (right) and concrete toe protection of delimiting wall in an urban river (left)

Objectives of an urban stream restoration project are the improvement of habitat quality, the flood control, protection of channel stability and prevention of erosion by environmentally sensitive solutions. Even if the return of the watercourse to natural conditions appears difficult and highly unlikely, structural and non-structural interventions can improve stream ecosystem. It is necessary to implement specific corridor enhancement and restoration activities that will solve existing problems or prevent future problems with regard to geomorphology, vegetation, and fish and wildlife habitat. Structural interventions constitute a direct approach to stream restoration, particularly in heavily urbanised conditions.

3. Structures for urban stream restoration

In urban water courses stream restoration structures vary from hard works to soft ones [2]. Generally they can be divided into the following groups:

- longitudinal structures, which are placed along the banks;
- full stream width structures spanning the stream and being overtopped by water under most of flow condition.

An accurate analysis of the context and boundary conditions should be done before proceeding with the design [3]. In particular the following aspects have to be considered:

- hydrological (evaluation of the intensity and duration of the rain, losses and infiltration, discharge at the generic watercourse section);
- hydraulic (evaluation of velocity and water depth);
- geotechnical (evaluation of the existing soil conditions including soil strength and permeability); and,
- geomorphologic processes (evaluation of erosion, sediment transport and sediment deposition).

3.1 LONGITUDINAL STRUCTURES

Longitudinal structures are used to protect river banks. The characteristics of urban streams and in particular the high erodibility of soils and the limited space available for protective measures, often suggest the use of hard structures (walls, sheet piles), which are difficult to remove and modify. Generally the interventions are confined to particular points such as the toe of the structure. If conditions permit and natural banks exist, the protection can be extended to the entire bankfull height. Rock protection or vegetation can be used.

In the following the most common structures for river restoration in the urban environment will be described.

3.1.1 Toe Protection

This intervention consists in placing rocks at the bank toe. It can be used when the higher part of the bank is stable enough, for example, when it is well protected by hard structures or vegetation.

The height of the toe protection should reach to between 1/3 and 2/3 of bank height. This kind of structure presents high flexibility, easy construction and easy maintenance. Longitudinal peaked stone toe protection and longitudinal stone fill can also be used (*Figure 4*). In the first case they are of a triangular form, with the lateral side disposed according to the angle of repose. This structure is usually used in incised streams, typically with sand beds. The second kind of intervention is a simple casting of rocks, in which rocks are placed adjacent to the banks or riverward of the high bank.

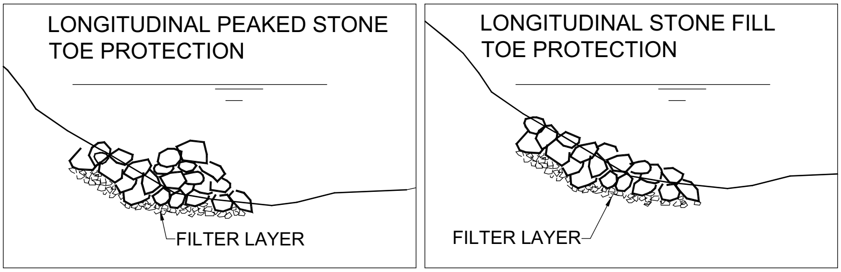

Figure 4. Longitudinal toe protection

Sometimes where the bank materials are highly erodible, and the adequacy of an unsupported stone placed along the toe of the bank may be marginal, stone dykes can be placed at intervals as "tiebacks" to prevent erosion from forming behind the structure. A spacing of one to two multiples of channel width can be used between tiebacks.

3.1.2 Bank protection

This kind of protection provides an immediate and effective protection against erosion, because it resists shear stresses at the bank. In the urban environment, longitudinal revetments are applied in reaches where bank instability or overtopping has dangerous consequences for the existing infrastructures. Sometimes they can be used for replacing traditional structures.

3.1.2.1Riprap

When flow velocity and shear stresses are high, stone revetment (also known as riprap) can be used (*Figure 5*). This type of restoration structure consists in lining the stream channel with a mixture of stones, in which the largest sizes resist hydraulic forces while the smaller ones prevent the loss of bank material and improve the interlocking among the largest stones. Riprap is relatively easy to install, is flexible and can be easily repaired, requires low maintenance and has a natural appearance. If the stones are well chosen a filter layer can be avoided, which ensures the permeability without removing bank material, and permits the growth of vegetation. Side slopes from 1:2 to 1:3 are recommended for riprap stability. To avoid the scour of the toe (that is the most common cause of failure) the lower end should be placed below the expected scour, or, stones can be placed over the bottom to avoid bed erosion near the bank. A riprap revetment is susceptible to displacement and deterioration of the rock placement. In this case the effectiveness of the structure is reduced and the revetment should be replaced as needed.

In mountain stream riprap protections extending across the entire cross section are used in order to protect the banks and riverbed.

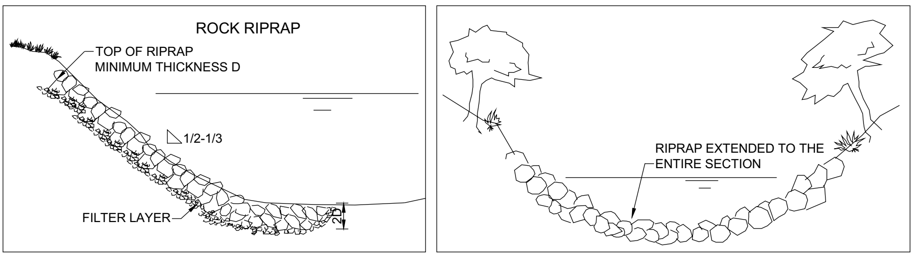

Figure 5. Bank protection with stone revetment and riprap extending over the entire cross section

3.1.2.2 Planting vegetation

The planting of vegetation represents a principal river restoration activity. The presence of plants on the banks allows improving their stability and, in conjunction with traditional structures, it reduces the environmental impact. Indigenous plants are preferred and can be planted as vegetative cuttings or using nursery-grown bare-rooted plants.

The use of vegetation in bank protection is recommended when flow velocity is not too high. In urban reaches the objective of using protective vegetation is to stop the erosion processes that occur on bare soils.

The type of vegetation and its speed of growth should be considered before planting. The measures such as seeding or planting bare-root and container-grown plants require long time before vegetation can contribute to the bank stabilisation and for this reason the plants should be used on the upper part of the bank or in low flow rivers. Urban streams are characterised by high energy flows and confined channels with limited or no adjacent flood plain, so in order to avoid losing vegetation during flood events, cuttings, pole plantings, and live stakes taken from species that sprout readily (e.g., willows) should be used in restoration works. Reliable sprouting properties, rapid growth, and general availability of cuttings of willows and other native species makes them particularly appropriate for use in bank re-vegetation projects, and consequently, these measures are used in most of the integrated bank protection approaches. The use of these measures also allows protecting banks from erosion by upslope water sources.

If bank does not suffer from soil movement, brush mattress [4] (*Figure 6*), in which a thick layer of branch cuttings is installed to cover and physically protect the stream bank, can be used for restoring riparian vegetation (if present) and stream bank habitat. This measure can be used in fast flowing streams.

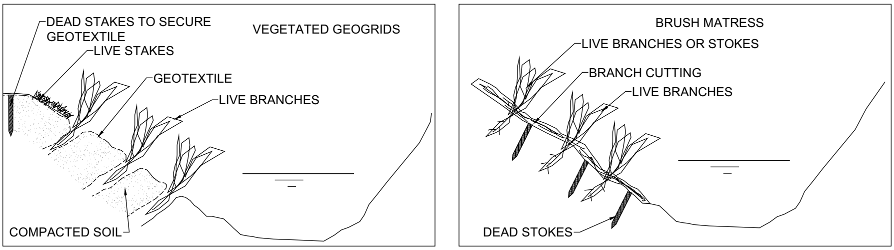

Figure 6. Bank protection with brush mattress (left) and vegetated geogrids (right)

In urban streams, vegetation is also applied to deal with specific ecological deficiencies caused by hard structures. For example, vegetative measures can be used in combination with stone toe protection. The design of these interventions must consider the stresses acting on vegetation. If they appear too high, other protective systems must be added to the vegetation (natural fabrics, wood, and rocks).

Geotextiles are used for erosion control on road embankments and other upland sections, usually in combination with seeding, or plants placed through slits in the fabric. Vegetated geogrids (*Figure 6*) are layers of live cuttings incorporated with natural or synthetic geogrids/geotextiles. Soils and granular backfills wrapped by geogrids/geotextiles alternate with layers of live cuttings.

3.2 STRUCTURES FOR GRADE CONTROL

Bed stability is one of the most important aspects for rivers in urban areas, because bed instability affects not only the bank but also the efficiency and functionality of works and structures of public interest (bridges, pipes). High flow velocities and the decline of sediment supply rates, which is caused by the progressive sealing of urban catchments, are responsible for erosion processes typical of urban streams. In the past, hard structures (concrete sills, concrete pavement) were normally applied to prevent bed instability resulting in negative effects on water quality and in-stream habitat.

In river restoration, structures for grade control span the full width of the stream and can be overtopped. They represent a fixed point in the bed, ensure an increase in the upstream water level, but may cause a scour on their downstream side. According to their geometric configuration they may or may not form an obstacle to fish migration.

Vegetation (particularly wood) or rocks can be used for construction of grade control structures, depending on shear stresses and material availability.

Rock structures should be used in straight reaches, in order to obtain a symmetric distribution of the stresses, which is a condition that cannot be ensured in meandering reaches, with dangerous consequences for the entire structure. These structures have to be installed in reaches with stable banks, and should extend into the bank for 1 or 2 metres. If banks are unstable it is necessary to use some of the earlier previously described measures. If the reach targeted for restoration is long enough, more small structures should be used instead of a few larger ones, in order to maintain the biological and morphological continuity of the water course. The distance between two consequent structures should be between 5-7 widths of the stream. The material used should be well selected and graded, to avoid the presence of interstices and the resulting formation of macro-roughness elements, which is a condition dangerous for the growth of macroinvertebrates.

The base material has to be well compacted while the upper part of the rock structure should represent a supplementary roughness increasing dissipation of flow energy.

In urban rivers low weirs, riffle/step and pool combinations, and block ramps can be used for grade control.

3.2.1 Low weirs

Low weirs are structures which can eliminate the near-bank forces, while maintaining channel capacity and grade of the bed [5]. They increase physical heterogeneity of the stream and permit creation of structural and hydraulic diversity in uniform channels. Weirs are used to collect and retain gravel for spawning habitat, deepen existing resting/jumping pools, create new pools above and/or below the structure, trap sediment, aerate water, and promote deposition of organic debris.

3.2.1.1 Boulder check dams

A boulder check dam (Figure 7) is a basic grade control structure that acts like a beaver dam in slow water and creates a "riffle". As a weir, it diminishes channel downcutting and supports the development of more stable channels. When constructed and spaced properly, check dams can simulate the natural pattern of pools and riffles occurring in undisturbed streams and the gravel deposits between such structures can be used by fish as spawning grounds. They also improve aquatic habitat by increasing dissolved oxygen in water. Check dams should be used in reaches with stable bed substrates, well developed and stable banks, and with low bedload transport, as often found in urban rivers. The structure should be embedded as far as possible into the streambed and should be anchored to a minimum of 1/3 of the stream width or 1.5-2.0 metres into the stream bank. Generally, a crest height placed 0.3 metres above the bed is sufficient for scour pool formation.

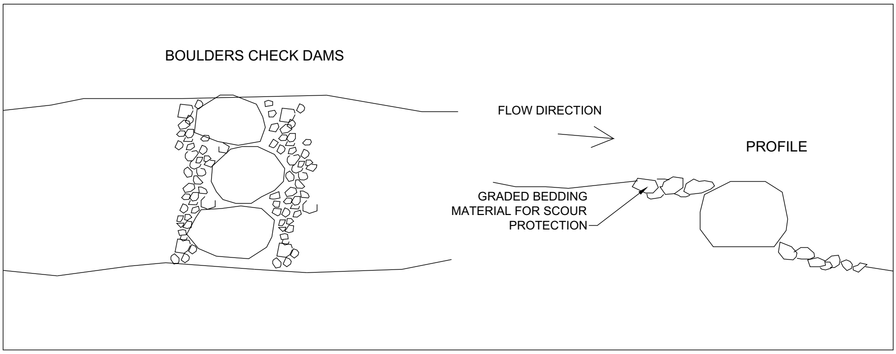

Figure 7. Boulder weir

Geotextile for scour prevention should be attached to the upstream portion of the structure, buried 0.3 metres into the streambed, and backfilled with adequately sized rocks. Once the excavated portion of the bank has been backfilled, it should be armoured with aptly sized riprap, sod mats, or willow transplants to prevent erosion and scour from compromising the integrity of the structure.

In mountain stream where wooden material can be easily found, logs can be used instead of boulders (*Figure 8*) even if they are more prone to failure because their downstream face is steeper than that of rock full-width structures and energy dissipation thus occurs in the scour hole. A preformed scour pool armoured with rock is necessary to provide protection against undercutting.

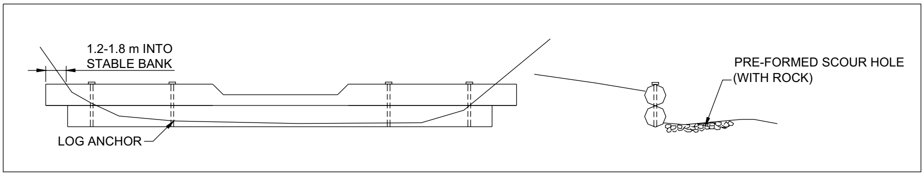

Figure 8. Log check dam

3.2.1.2 Rock weir

Rock weirs [6] are grade control structures designed to reduce the energy of a stream that would otherwise contribute to stream bank erosion. These devices are appropriate in areas of high gradients typical of mountain reaches. The rock weirs will provide a mid-channel scour pool, which is necessary for improving fish habitat. The function of this type of structures is to concentrate the effects of a large elevation change in the stream channel into a more controlled situation, while at the same time allowing fish passage and formation of a low-flow stream channel. Different forms of rock weirs can be used in stream restoration (*Figure 9*).

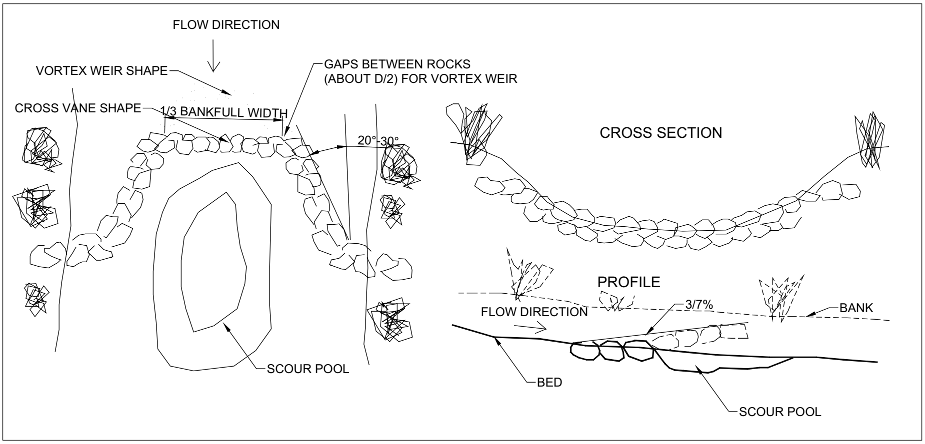

Figure 9. **Rock weir**

Cross vanes are typically designed with a "U" shape and oriented so that the apex of the structure points upstream. The stone is trenched into the stream bank at sharp angles in a general "V" shape pointing upstream. Vortex rock weirs resemble modified horseshoe shapes, again with the apex of the structure pointing upstream. The top elevation of the centre vortex rock(s) at the apex of the weir should be at or near the bed level to permit fish passage at low flows. Gaps between rocks are located between 1/3 and 2/3 of the stream width. W-weir installation should proceed similarly to the vortex weir construction and should account for the more complicated geometry of the structure. It consists of two adjacent weirs. This configuration provides a mid-channel on the upstream side, and scour pools below the weir for fish habitat, resting, and acceleration manoeuvres during fish passage.

3.2.2 Riffle and pools

The structures described above can be used to reproduce the natural riffle formations, which are present in streams with gradients 0.0015-0.005. Riffle and pool structures [4] comprise a series of relatively quiescent pools separated by fast stretches of relatively shallow flows (riffle). They can be used to replace concrete bed revetment, giving the water course ideal conditions for improving fisheries. Riffle and pools should be designed to permit fish passage during low flow condition. In particular the permeability of the structure must avoid the percolation of water without overtopping the structure. In urban reaches permanent riffles are used: they are constructed with rocks of the dimensions necessary to resist shear stresses. Sometimes the use of rock of larger dimensions is useful to increase the stability of the structure and to increase the flow regime complexity for broadening the range of habitat conditions.

Downstream riffle slope should be less than 1:20, while the upstream slope is a function of the angle of repose of the material, but less than 1:4. The central part of the riffle should be designed lower than the lateral ones, for example with a V shape. In this way low flows are concentrated, which is a great advantage for fisheries.

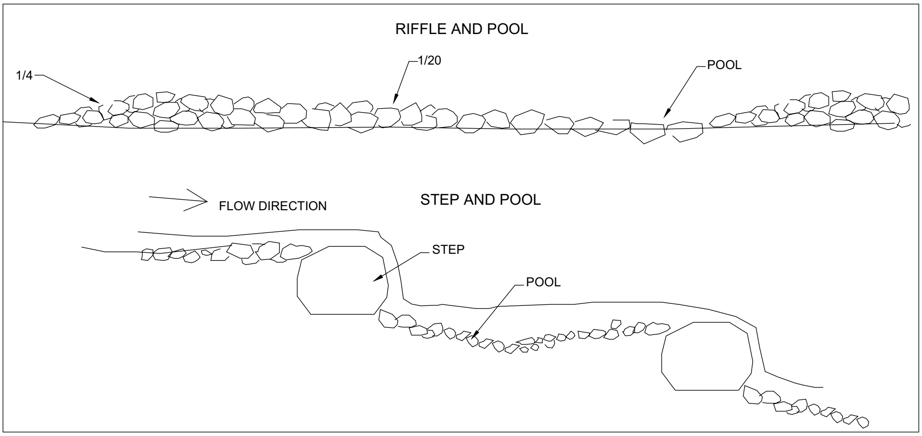

Figure 10. Permanent riffle and pool and boulder check dams in a sequence of step and pool

3.2.3 Step and pools

In mountain reaches step and pool stream restoration can be used [7, 8]. They are typically associated with well confined (urban), high-gradient channels with slopes greater than 3%, having small width to depth ratios, and bed material dominated by cobbles and boulders [9]. Step pools generally function as grade control structures and aquatic habitat features by reducing channel gradients and promoting flow diversity. They are characterised by a succession of channel-spanning steps formed by large grouped boulders that separate pools containing finer rock (*Figure 10*).

Step-pool units should be designed and constructed to have a characteristic step height, H, and step length, L, and all steps should be firmly anchored into the stream bank. Step rocks are placed on footer rocks so that they rest on two halves of each footer rock below, so that the step rock is offset in the upstream direction. Footer rocks should extend below the scour hole elevation.

3.2.4 Block ramp

Block ramps (*Figure 11*) are naturalistic stream restoration structures used to convey water to a lower elevation with high dissipation of energy. They represent an alternative method of protecting soil surface and maintaining a stable slope. Ramps maintain the morphologic continuity ensuring a biological exchange between upstream and downstream parts, with an excellent environmental enhancement. The structures are flexible and not hard. If necessary a fish passage can be inserted in the ramp. Ramp can be well used in urban reaches, where streams are straight and confined. River bed should be sufficiently stable in order to avoid differential settlements and to have uniform conditions all over the ramp. Block ramp can be used both in mountain and plains reaches, with the gradient less than 10%. The slope of a ramp varies from 1:4 to 1:12. Rocks can be placed in a regular form or loaded in bulk. A filter layer can be used if bed material is too small, with respect to rock dimensions. A geotexile should be wrapped around the ramp base.

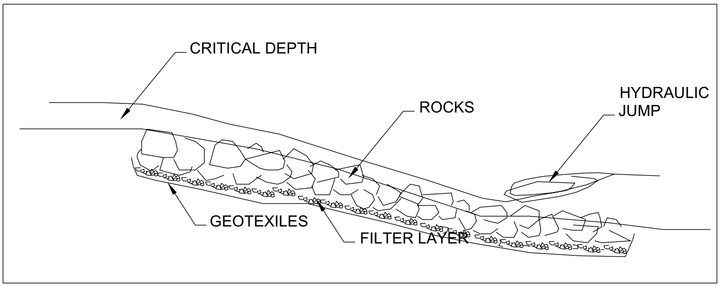

Figure 11. Block ramp

In urban streams block ramps are usually designed with uniform geometry, because of their low elevation. If it is necessary to increase ramp roughness, the ramp can be partially covered with boulders. From the hydraulic point of view, ramp is a steep reach preceded and followed by the mild ones. At the upstream end of the ramp the critical depth occurs while at the downstream end, a hydraulic jump occurs. The energy dissipation increases with the decline of the slope and the differences tend to vanish with decreasing discharges. If E_0 is the upstream water energy and E_1 denotes the energy at the toe of the ramp, the rate of energy dissipation is related to the height of the ramp D, the critical depth k and the slope of the ramp α by the following expression valid for very rough ramps (Pagliara and Dazzini [10]):

$$\frac{E_0 - E_1}{E_0} = 1 - \frac{(0.925\alpha + 1.931)\dfrac{k}{D}.}{(10.595\alpha^2 - 6.817\alpha + 2.417)\dfrac{k}{D} + 1}$$

Block ramps cause higher energy dissipation than the traditional structures (sills, check dams) and their stilling pool, required to contain hydraulic jump, is shorter. If boulders are used they should be placed on the downstream face of the ramp, because from a hydraulic point of view, the effect on the dissipation and the position of the hydraulic jump is greater [11]. These effects increase with the increasing discharge and the decreasing slope. In the design of the ramp, stability of rocks must be ensured. There are formulas expressing the relations among the diameter of the rock used, the unit discharge and the slope of the ramp, needed to ensure the stability. For material loaded in bulk with a ramp height greater than 2 m, the following expression can be used (Whittaker [12]):

$$q = 0.257\sqrt{\frac{\gamma_s - \gamma_w}{\gamma_w}g}\,\alpha^{-7/6} d_{65}^{3/2}$$

where q is the critical unit discharge, g_s is specific weight of rocks, g_w is water specific weight, α is the slope of the ramp and d_{65} is the nominal diameter of the material for which 65% of the sample is finer by weight.

Another expression determined for rock chute channel stability is the following (Robinson and al. [13]):

$$q = (D_{50}S)^{1.40+0.213/S^{0.5}}\, e^{(-11.2+1.46/S^{0.5})}$$

where D_{50} is rock size for which 50% of the sample is finer by weight and S is the slope. The critical part of ramp stability is the downstream half, in which the shear stresses are greater.

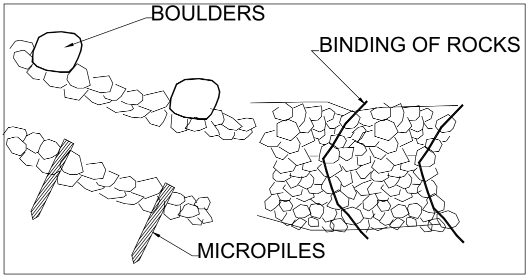

Figure 12. Stabilisation of a block ramp (left) and experimental setup (right)

Experimental tests showed that ramp failure is sudden and the movement of few elements leads to the total failure [14]. It possible to increase the ramp stability using micropiles uniformly placed on the ramp, anchoring the rocks with steel wires, or placing boulders over the ramp (*Figure 12*). In this last case the tests were conducted on ramps of different slopes (1:4, 1:8 and 1:12) covered with rocks having medium diameters of 11.25, 22.05 and 32 mm, where boulders (having a diameter three times those of the base material) in different concentrations were placed (*Figure 12*). Boulders projected from the base material about half a diameter. Boulders placed in rows, random and in arch forms were examined, in order to evaluate what is the best choice for the design of these structures. In particular arch forms appear less stable than other forms because water flow is concentrated at the centre of the flume, while there are not evident differences between the random disposition or placement in rows. The increase in stability reaches values up to 60% with respect to the base condition.

3.3 OUTLET PROTECTION

In urban areas pipes and culverts are very numerous. Traditional protection consists of a concrete revetment which does not permit the growth of vegetation and biological exchange with the surrounding terrain. To improve aquatic habitat, rock outlet protection can be used (*Figure 13*). This intervention consists in placing rocks at the culvert outlets, conduits and pipes, with the purpose of preventing scour erosion, minimising the downstream erosion, reducing the flow velocity and the water energy, and reducing the effect of turbidity. Treatment should be extended to the point where flow velocity and energy are dissipated to the degree where there is a minimal risk of erosion of the receiving channel. The minimum thickness of the riprap should be 1.5 times the maximum stone diameter. The apron should not have a drop at the end, but rather the top and the receiving bed should be aligned at the same level. If the water discharge is directed into a well defined channel, the apron must be extended across the channel bottom and up the channel banks. Generally this kind of intervention can be used where outflow speeds exceed 3 m/s. For low flows, vegetation, including sod, can be used instead of the rocks.

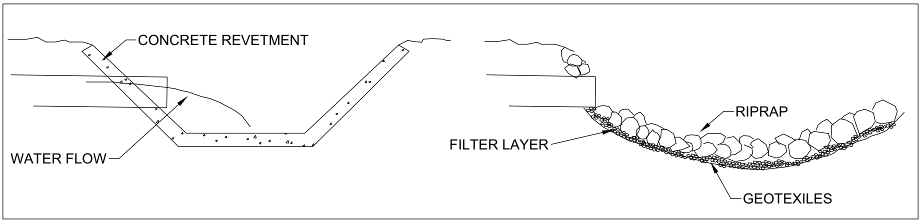

Figure 13. Outlet rock protection

4. The adaptation of existing structures

In a river restoration it is often necessary the adapt existing structures, without their complete demolition that would increase the costs. The adaptation of an existing structures to a design, which is more environmentally friendly, is particularly attractive, if the original structure requires restoration. One kind of intervention consists in modifying a concrete sill or weir in the block ramp (*Figure 14*).

Rocks can be placed downstream of the existing structure, reducing thereby the height of the sill required to join the ramp and the stream bed. If long periods of low flow occur, the ramp can be placed laterally, creating a fish passage. Blocks should be placed randomly to distribute the flow and create an alternating pattern of subcritical and supercritical flows.

The presence of ramp modifies the hydraulic regime of flow over the existing weir and its rating curve, and increases the water surface elevation for a given discharge by 25 to 30% [15]. This aspect must be investigated especially in urban rivers, where flood discharge must remain confined to the river cross section.

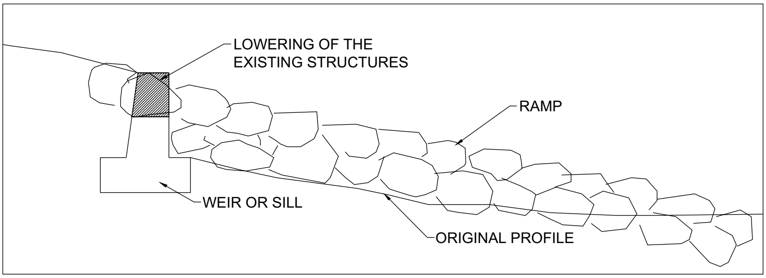

Figure 14. Incorporation of a concrete weir into a block ramp

5. Conclusions

Urban stream restoration is important for improving habitat quality and aquatic ecosystems, which are subject to continuous degradation due to urbanisation and its consequences. Different structures can be used to control the changes and the dynamics of streams with environmentally friendly solutions. The choice must be based on the stream and watershed characteristics. Restoration structures must be monitored and continuously adapted to changing conditions in order to attain the chosen restoration objectives.

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RESTORATION OF URBAN RIVER HABITAT IN COMPLIANCE WITH EU DIRECTIVES

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1. Introduction

The adoption and implementation of the Directive 2000/60/EC [1] with respect to crucial changes in the approach to protection of waters and requirement to unify methods, which are used for monitoring, assessing and sharing of information among member countries of EC, create a lot of problems. These problems are solved in join research programs of the member and candidate countries. Considering the specific ecological conditions of watersheds in individual countries, each country has to use its own monitoring data and create a program and suggest a classification system for assessing ecological status. The monitoring and research results from individual countries will be adjusted by calibration of single countries during the transient period until the Directive would be valid in the Union.

The assessment of the ecological status of a stream is very complex process, which has to be done according to the Directive 2000/60/EC [1]. The Directive requires a complex study of the stream ecosystem by monitoring not only water and sediment quality, but also studying the status of aquatic organisms and observing changes in the morphology and hydraulics of the stream.

The basis for the classification of a chemical status of a water body and for the assessment of ecological risk is an environmental quality standard (EQS), which is according to the Directive 2000/60/EC [1] defined as the concentration of a polluting substance or a group of substances in water, sediment, or live organisms, and which should not be exceeded to protect human health and the environment. It is very difficult to assess EQSs for various pollutants, because we have to not only carry out the research program, but also discuss results with, and gain their acceptance by, a wide spectrum of specialists and involve the public in this process. This overall process becomes even more difficult because of the lack of EQS limiting values in the Czech and European legislatives, especially for aquatic sediments.

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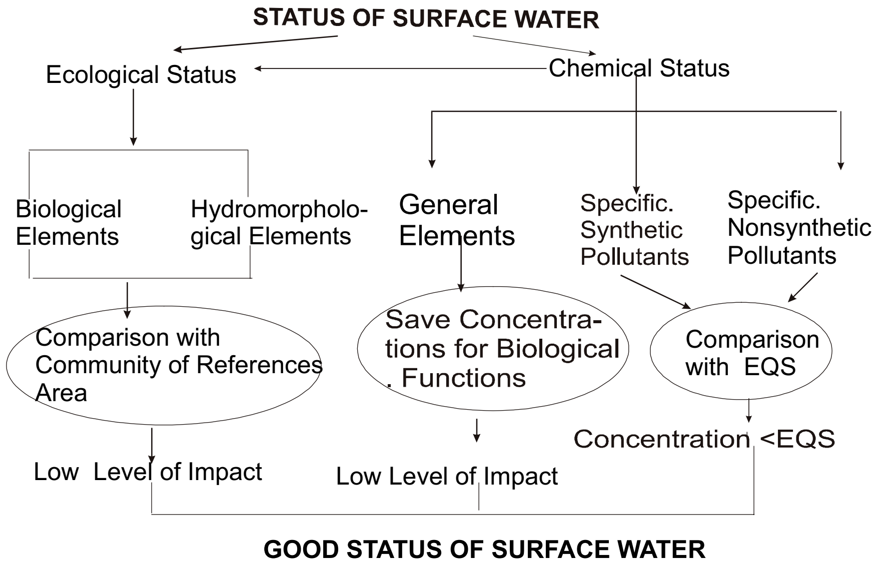

Figure 1. Surface water status assessment, with respect to achieving at least good status of surface waters (according to Directive 2000/60/EC) (The left part of figure shows a new approach to the surface waters status assessment).

Assessment of the river status is undergoing historical development. It started by evaluation of water quality by chemical and physical parameters only. Later on, microbiological analyses were added and in some countries evaluation of the saprobic index and the level of eutrophication and acidification was also used. These methods are still used, but they are not sufficient for assessing the ecological status. They are indicating one factor only (organic pollution-saprobic index, nutrients-level of eutrophication) and they are useful only in limited geographic areas and for particular types of rivers and streams.

The new approach to the assessment of water quality requires comparisons with reference localities. This new approach was a reason why during the 5th RTD program there was a requirement to develop uniform methodology to assess the ecological status of running waters and to generally define the target status at the European scale (project AQEM, [2]. Recently, the process of selection reference localities progressed in Europe. The first stage of this process focuses on selecting reference sites for rivers and streams, which are minimally affected by human activities. This information is used for developing different types of computer models. Computer models are used for the description of the aquatic organism environment.

For example, the British programs RIVPAC [3] and Czech PERLA/HOBENT [4] are used for assessing the biological quality of rivers and streams based on the macroinvertebrate communities. They also assess the impact of anthropogenic activities on streams, and allow the comparison of existing benthic communities with a “target” community, which would exist in the absence of any impacts. This comparison is made by comparing basic environmental parameters (altitude, substrate, flow, etc.) of a particular site with a basic database. The basic database includes sites which are not impacted, or have very low impacts by human activities. Thus, it is possible to find out how far a particular community deviates from the natural state. All these comparisons are based on natural conditions. At present, data for sites in urban watersheds are completely missing. Because it is not possible to find non-impacted streams in urban areas, it would be advantageous to choose as a reference a stream that is impacted, but whose conditions have stabilised at some equilibrium. Then one could observe its development in new conditions. Generally we can say that the main disadvantage of these models is the fact that they are still not very useful in the case of urban streams.

2. Case study

2.1 EXPERIMENTAL CATCHMENTS

The first assessed stream is Botič Creek, the largest tributary of the Vltava River in the Prague agglomeration. Its length is 34.5 km and watershed area 134.84 km^2. A study section of the stream is below the Hostivař reservoir, from km 10.719 to km 12.745. In the upstream drainage area, agricultural activities represent the main source of pollution, but in the study sections (sampling sites from B0 to B5 shown in the figure bellow) point sources of pollution are more important. There are two overflows (CSO1 and CSO2) from the combined sewer system (at the sampling sites B2 and B4) and one storm sewer outlet (SSO) at the sampling site B3 (Figure 2) in the study reach. The quality of wastewater from the overflows is influenced by the industry, which contributes loads of Cu, Zn, and Ni, but not much of Cd and Pb.

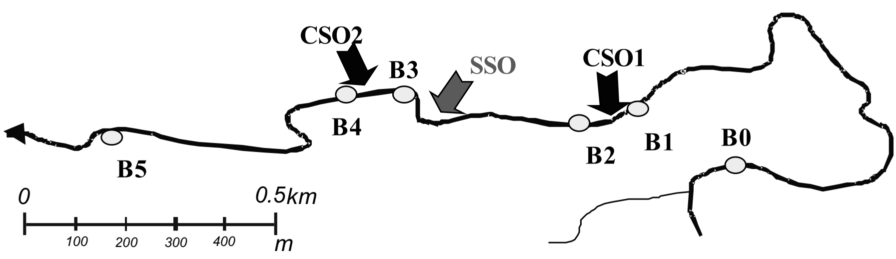

Figure 2. Botič Creek: study reach

The second assessed stream was Zátišský Creek, a tributary of the Vltava River with the total length of 3.080 km, total drainage area of 3.022 km^2 and the natural flow at the stream outlet of Q_5=1.9 m^3/s. The difference in elevation between the spring and the outlet is 83 metres. The stream runs mainly through built-up areas (residential areas) with natural floodplains of various sizes. Besides an outdoor swimming pool at the stream spring, there are three storm sedimentation basins on this stream (SSB). The storm sewer system with 8 outlets (SSO) is the main source of water pollution. Because of these outlets, natural flow in the stream greatly increases during rain events. Such high flows then cause strong devastation (erosion) of the creek. Zátišský creek was monitored at seven sampling sites (Figure 3), which were chosen to assess the storm sewer discharge impacts and the flow retention in sedimentation basins.

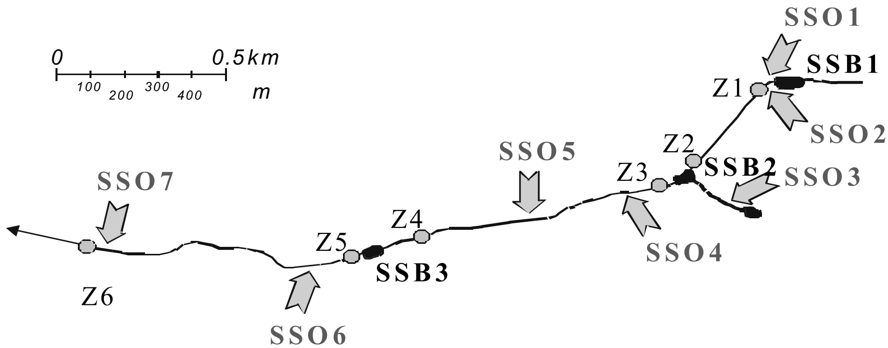

Figure 3. Zátišský creek: study reach

The Komořanský creek (KO) with a total length 3.613 km and drainage area of 3.5 km^2 is also a tributary of the Vltava River. It runs largely through a wooded area and it is considered to be unaffected by anthropogenic activities. A sampling site with similar conditions (but without sewer impacts) to those in Botič creek and Zátišský creek was chosen as a reference site.

2.2 METHODS

2.2.1 Hydraulics

The assessment of urban drainage impacts on stream hydrology was done by runoff simulations and the monitoring of rain events and discharges in the sewer system.

2.2.2 Chemistry

The chemical analyses were limited to basic chemical parameters, which were extended for heavy metal (HM) analyses. Concentrations of heavy metals were measured not only in the water phase, but also in sediment and in biomass of aquatic invertebrates. Heavy metal concentrations were also measured in the sediment and water discharged from the combined sewer system. The HM concentrations were measured in sediment and organisms after they were digested in microwave in the presence of HNO_3 a H_2O_2 [5, 6].

2.2.3 Risk assessment

The potential environmental risk was evaluated by using three parameters: the partitioning coefficient, hazard quotient and metal pollution index.

2.2.3.1 Distribution coefficient

The partitioning coefficient (Kd) relates the metal concentration in sediment to that in water (typically in units of ml/g) and identifies which medium (water or the solid phase of sediment) is crucial for the risk assessment, i.e., if pollutant prefers binding to sediment or dissolution in water [7].

2.2.3.2 Hazard Quotient

The Hazard Quotient (HQ) is calculated according to Barnthouse [8] as:

$$HQ = \frac{C_M}{TC} \textit{ (dimensionless)}$$

where C_M represents the metal concentration in the stream sediment and TC represents a toxicological criterion. When HQ value exceeds 1 (for a particular metal), an ecological risk is identified. If the risk for five heavy metals has been assessed together, an increased risk occurs when HQ exceeds the value of 5**.** For the assessment of metal concentrations in water, the Czech toxicological criteria were used (the 3rd class from the regulation CSN 75 7221, [9]). For calculation of HQ in bottom sediment, the US EPA toxicological benchmarkers TEC (Threshold Effect Concentration) and PEC (Probable Effect Concentration) [10] were used, because Czech criteria for sediment evaluation are missing.

2.2.3.3 Metal Pollution Index

The Metal Pollution Index (MPI) evaluates the overall risk of observed metals and is defined as:

$$MPI = \sum C_{Fi} \frac{W_i}{W_T} \textit{ (dimensionless)}$$

Where W_i is weight of metal *i*, W_T is ΣW_i and C_{Fi} is a contamination factor for metal *i* expressing the relation between the measured metal content at any given site and the background level which has been measured in sediment samples from unpolluted reaches of the Komořanský creek, as proposed by Goncalves et al. [11, 12]. Weighting is based on relative toxicity of different metals and is calculated according to Goncalves et al. [11] using criteria PEC and the 3rd class of CSN 75 7221 [9]. The site is classified as unpolluted in the case of the five observed metals if MPI ≤ 1.

2.2.4 Biological assessment

We have used two types of biotic indexes for biological assessment. Assessment of localities is based on occurrence of individual species or groups and indicates values, which accrue from social diversity. The saprobic index expresses water pollution by organic material and its influence on water associated biota. The saprobic index is calculated according to the Czech standard (CSN 75 7716 [13]). A second biotic index type is the BMWP score system [4]. This index is calculated as a total sum of individual scores of the found taxon (per family, in this case). If we divide the BMWP value by the scored family number we obtain the ASPT index [4]. An advantage of this index is its independence on the systematic taxon number found in the sample. A higher value of both these indices (BMWP and ASPT) means a healthier environment.

2.3 RESULTS

2.3.1 Hydraulics

In the case of Botič creek, runoff discharged from CSO1 shows a significant impact on the stream. Number of overflows from CSO1 during one year is smaller than from CSO2, but the total volume of water spilled from CSO1 is three time greater than that from CSO2 (Table 1). The maximum annual discharge from CSO1 exceeds that from CSO2 six times.

Table 1. Results of dynamic simulation [14]

Results of dynamic simulation	CSO1	CSO2
Number of overflows during one year	18	34
Total volume of spilled water (m^3)	19 027	6 763
Peak discharge (m^3/s)	2.014	0.352

It is significant that CSO1 causes a hydraulic stress on the stream. The impact of CSO2 is not too large. It is probable that the greatest hydraulic stress occurs during summer storm events, when the discharge in the stream rapidly increases from annual minimum values to the annual maximum values.

2.3.2 Chemistry

The water quality is degraded by high concentrations of nitrate and nitrite anions at all observed sampling sites. These substances indicate agricultural or faecal pollution.

The Kd values for the observed metals have been measured within the range from 10^4 ml/g (for Cd) to 10^5 ml/g (for Zn). Thus, the heavy metals are preferably bound to sediment (solid phase).

The concentrations of HM in water and sediment of Komořanský creek did not reach the critical values, which would indicate an ecological risk. The measured concentrations of heavy metals (below the 90th percentile) in water of Botič creek indicate no ecological risk according to HQ and a very slightly increased risk according to MPI.

The heavy metals concentrations in water of Zátišský creek are not at dangerous levels, from a long-term view; in most of cases they have been below the detection limits of the graphite furnace atomic absorption spectrometer (GF AAS) (0.0002 mg/l for Cd, 0.07 mg/l for Cu, 0.002 mg/l for Ni and Pb).

The HM concentrations were measured not only in the stream sediment, but also in the sewer sediment. Table 2 shows that there are significant differences between the burdens of HM in stream and sewer sediments. During rain events, the sewer sediment is washed out into the stream and there it mixes with stream sediment, which increases the stream sediment pollution [15].

Table 2. Heavy metal concentrations in sewer and stream sediment (mg/kg dw)

	Sewer	B0	B2	B3	B4
Cd	0.98	1.03	1.23	1.15	1.36
Cr	374	19.03	353.9	48.26	61.01
Cu	517	18.32	493.66	194.33	226.4
Ni	157.8	17.02	154.56	31.5	49.13
Pb	126.38	20.45	103.59	127.8	250
Zn	574.47	116.13	340.32	365.93	493.28

The risk assessment of heavy metals in sediment of the Botič creek by HQ (using TEC and PEC criteria) indicates significant negative ecological impact on stream environment, as also shown in Figure 4. CSO influence on increased sediment toxicity is evident. Whereas Cu toxicity dominates below CSO1 (B2 profile), Pb is the main pollutant in profile B4, impacted by CSO2 as well as SSO. The monitored heavy metals are dangerous to the stream almost in all experimental profiles according to both used criteria; according to TEC the risk is very significant.

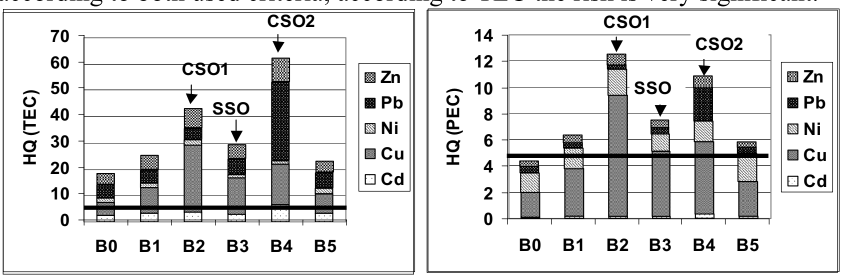

Figure 4: Hazard quotient for sediment of the Botič creek, based on TEC and PEC criteria. HQ>5 indicates ecological risk

The risk assessment of heavy metals in sediment of the Botič creek according to MPI shows similar results as the assessment by HQ, see Figure 5 (in this case MPI≥1 indicates the risk). However, high weight is adjusted to Cd because of very low limit values CSN as well as PEC and that is why Cd appears as the major contaminant in all observed profiles [16].

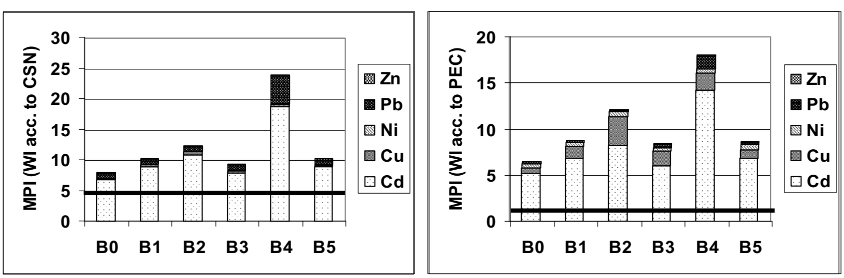

Figure 5: Metal pollution index for sediment of the Botič creek, weight (W_i) has been calculated according to the 3rd class criteria of CSN 75 7221 and PEC. MPI≥1 indicates anthropogenic impact.

In case of Zátišský creek, concentrations of monitored heavy metals in sediment of this stream can create only slightly increased risk in profile Z1 due to higher Cu concentration (probably caused by Cu roofs and drips in the surrounding housing estate). A negative impact of storm sedimentation basin has been observed in the case of SSB3; the metals concentrations in sediment below SSB (profile Z5) are higher than above it (profile Z4). Wash out of the pollution deposited in basin sediment is probable primary cause of this fact [16]. The assessment according to HQ shows that estimation of the risk depends on the selection of the criterion. In the case of the stricter criterion TEC (left graph in Figure 6) total risk of the monitored metals in profile Z1 is increased, while in the case of PEC it is not (right graph in Figure 6). This statement proves the necessity of application of more than one criterion in HQ evaluation, as recommended by Jones [10].

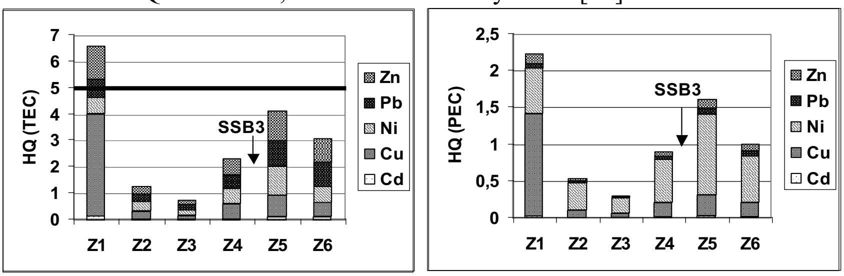

Figure 6: Hazard quotient for sediment of Zátišský creek, based on TEC and PEC criteria. HQ>5 indicates ecological risk.

The assessment based on MPI (Figure 7) shows in the Zátišský creek similar results as in Botič creek. Cd is dominant in the MPI value especially if W_i is established according to the criterion of CSN.

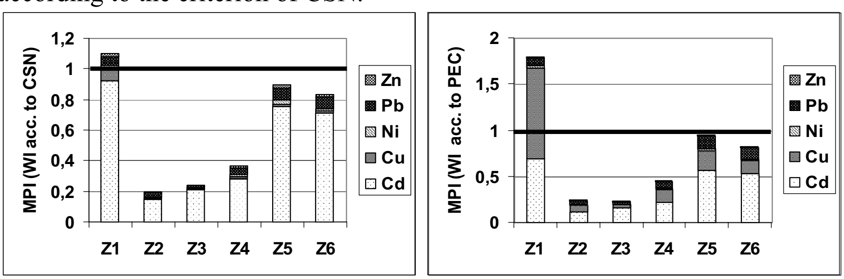

Figure 7: Metal pollution index for sediment of Zátišský creek, weight (W_i) has been calculated according to criteria of the 3rd class of CSN 75 7221 and PEC. MPI≥1 indicates an anthropogenic impact.

Considering the fact that monitored sites are impacted by human activities to different extent, the composition of benthic community alters from site to site, which means that it is difficult to compare loadings of organisms from different sites. For this reason we choose organisms which were present at most sites, for example *Hydropsychidae*. Figure 8 shows variation among profiles, where *Hydropsychidae* were present. The results show that *Hydropsychidae* accumulated higher levels of heavy metals in Zátišský creek than in Botič creek, where the pollution of water and sediment was higher. We can assume from this fact that in Zátišský creek the elements were present in more bioavailable forms than in Botič creek.

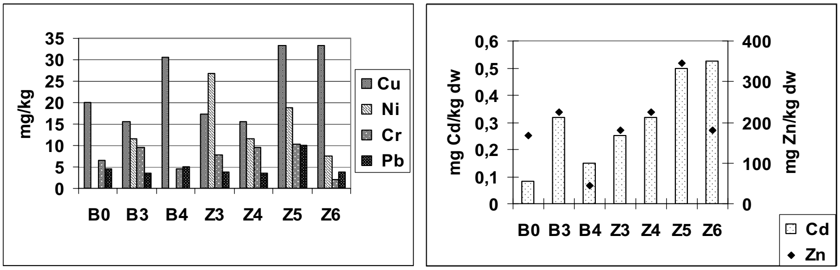

Figure 8: Concentration of Cu, Ni, Cr, Pb, Cd and Zn in body tissue of *Hydropsychidae* sp.

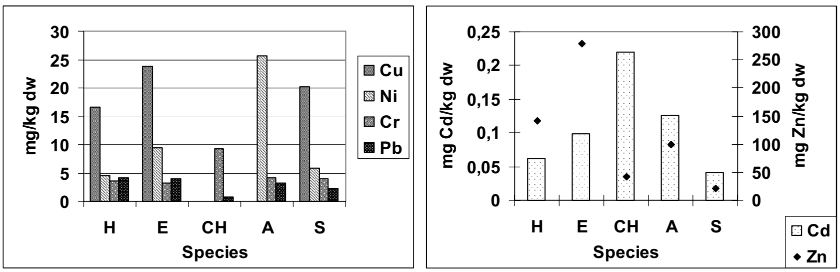

Figure 9: Concentration of Cr, Ni, Cu, Pb, Cd and Zn in body tissue of all present species at site B3. H-*Hydropsychidae*, E-*Erpobdellidae*, CH-*Chironomidae*, A-*Asellidae*, S-*Sphaeridae*

Figure 9 presents metal burdens in different species from one sampling site. The differences among species can be explained mostly by different feeding habits, which is in agreement with results of van Hattum et al [17], Kiffney et al [18], Farag et al [19], and Komínková et al. [20].

Anthropogenic influences on water quality and ecosystems conditions were ascertained also by biological assessment. As we assumed the best quality was observed at the reference site KO. An interesting phenomenon has been observed in profile B1, where a high number of individuals have been monitored; diversity in this profile is higher than in Komořanský creek (Table 3). It can be explained by reasoning that the same increase of organic and inorganic materials can produce environmental expansion and facilitate the presence of a larger number of species. The CSO influence on stream biology (profiles B2 and B4) is evident; it is indicated by decrease of ASPT values.

Table3. Biological assessment

Site	Description	Total n. of organisms	N. of taxons	SI	BMWP	ASPT	Quality class according SI	Quality class according ASPT
KO	Reference	1125	5	0.92	32	6.4	1	2
B1	Above CSO1	257	9	2.64	32	4.0	3	4
B2	Below CSO1	31	6	2.42	18	3.6	3	4
B3	Below SSO	69	8	2.43	27	3.9	3	4
B4	Below CSO2	85	5	2.39	18	3.6	3	4
Z1	Below SSO1,2	45	3	3.2	6	2.0	4	5
Z2	Above SSB2	29	3	2.49	9	3.0	3	4
Z3	Below SSB2	25	4	2.63	11	2.8	3	4
Z4	Below SSO4,5	11	2	3.22	5	2.5	4	5
Z5	Below SSB3	64	6	2.36	17	3.4	3	4
Z6	Below SSO6	96	7	2.24	21	3.0	3	4

Table 3 also shows that reference sampling site (KO), which represents a locality relatively unaffected by anthropogenic activity (flow, pollution), has quite different biology than Zátišský creek affected by storm drainage. Because of that, decrease of total number of organisms and taxons was observed. There was a slight decrease of number of individuals below SSB2, however, the taxon (family) number increased. This phenomenon is stronger below SSB3. We can say that it is caused by a longtime improvement of water quality below the storm sedimentation basins despite of the results of heavy metals risk assessment.

3. Concluding Remarks

This paper tried to demonstrate a complex approach to the ecological risk assessment in small urban streams and has showed the importance of combination of more indicators and criteria in the evaluation procedure. Only a multiparametrical approach can provide some idea about the actual status and ecological risk in small urban streams. The method of assessment presented in this paper is not comprehensive, but can show the main problems of each small urban stream and can form a basis of the risk quantification and the total ecological assessment according to the requirements of Directive 2000/60/EU.

The main facts of the assessment of the two Prague streams are:

- Heavy metals concentrations in surface waters do not represent any ecological risk from the long-term view in both assessed streams,
- The load of heavy metals in bottom sediment of Botič creek seems to be long-term; increased ecological risk of sediment has been observed at all sampling sites; Cu toxicity has been dominant below CSOs; Pb toxicity has occurred mainly below the storm sewer system outlet,
- Combined sewer system deteriorates water and sediment quality and quantity and also biological parameters in Botič creek,
- In Zátišský creek the storm sedimentation basin (SSB3) increases heavy metal concentrations in the effluent water,

- Hydraulic stress caused by CSOs creates morphological modifications and macrozoobentos structure changes in Zátišský creek.

The results of this study were used by the manager of the sewer system (Prague Water Supply and Sewerage Company) and manager of the stream authority (Forests of the capital region of Prague) as background for technical decisions and changes of stream management, which will be included in the Master Plan of Prague. The toxicological effect of CSO1 on Botič creek is significant, due to the low flow capacity of the sewer reach contributing to CSO1 (the dilution ration is 1:1.6). During the future reconstruction of sewer system the sewer capacity will increase, and so will the dilution ration, to 1:8 in the first stage, and finally to 1:16. Under such circumstances, the toxicological effect will not be significant at all.

4. Acknowledgements

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URBAN STREAMS – URBAN MASTER PLANNING AND HEAVILY MODIFIED WATER BODIES DESIGNATION REQUIRED BY WFD

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1. Introduction

The Czech Republic is expected to accede to the European Union in 2004, and the government is therefore committed to harmonise its legislation with EU, with all the economic, institutional, technical and financial implications involved. The paper presents authors' understanding of application of the Water Framework Directive (WFD) to Urban Areas, where the master planning approach has been successfully applied in the past decades in several places. Stream restoration activities in urban areas have to be integrated into a clearly defined framework which is understood as the Urban Drainage Master Plan (UDMP) or more specifically the Master Plan of Urban Streams (MPUS) as a part of UDMP. From experience of the team it is clear that such activity has so far nearly no relation to WFD and more specifically to the newly introduced activities, which are defined in WFD.

There are many water bodies that have been substantially changed by human activities. Clearly it is not appropriate to aim for the achievement of good ecological status in these bodies. Nevertheless, it is desirable to improve the ecological quality of these water bodies. The Water Framework Directive recognises this problem. The objective set for these bodies is a good ecological potential, rather than a good surface water status.

For surface waters the overall goal of the WFD for the member states is to achieve "good ecological and chemical status" in all bodies of surface water by the year 2015. Some water bodies may not achieve these objectives for several reasons. Under certain conditions the WFD permits Member states to identify and designate artificial water bodies (AWB) and heavily modified water bodies (HMWB) according to the article 4(3) of the WFD. HMWB are the bodies of water, which as a result of physical alteration by human activities, are substantially changed in character and can not therefore meet the good ecological status. AWB are water bodies created by human activities.

The authors have good experience with Master Planning (MP) of urban streams, where the restoration activities and environmental protection were main drivers for setting up the performance indicators for the urban drainage system. The application of the new legislation, including such issues as designation of water bodies, can introduce a different point of view of urban streams restoration.

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Application of WFD with all aspects in the basin may create a potential conflict with the already established restoration activities on urban streams. These streams must be designated as the water bodies and these bodies will be further designated as HMWB, AWB and/or natural streams. Some of the tributaries of urban streams with their upper reaches outside of urban areas have other status, because they are more natural. The authors will aim to show a potential conflict between the normal approach to MPUS and the newly requested designation of water bodies.

There are some basic steps in the methodology of urban stream master planning:

- Integrated solutions
- Application of simulation tools, and
- Digital processing of Master plans.

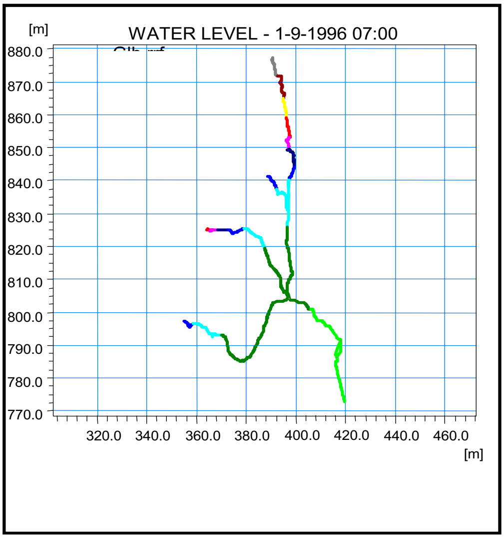
Modern Master Plans of urban streams show the new, in many ways different, view of problems and create new possibilities of solutions. This new approach deals not only with the differences in the complexity of approaches, but especially it serves a new methodology of problem solutions, technology and project outputs.

There have been very important changes, even in the basic concepts and technical solutions, in the field of designing, evaluating and implementing urban drainage systems during the last few years.

The original philosophy of urban drainage is changing. The original idea of effectively and safely removing all wastewater as fast as possible to the wastewater treatment plant or to the receiving waters is now changing to the idea of returning to, as much as possible, natural runoff conditions in the catchments, in other words to a sustainable natural status.

Figure 1A. Master planning for an urban stream

To develop a common understanding of the most effective approach to the identification of significant anthropogenic pressures on a river basin and the analysis of a potential impact of such pressures is to fulfil one of the basic requirements of WFD (Annex II). It is essential to identify appropriate tools (such as models, DSS, database, monitoring networks) in order to carry out the necessary analysis and to identify where new tools have to be developed or the current ones improved. Based on these findings the HMWB methodology will be completed and recommended. On the other hand the experience gained from master plans based on the Hydroinformatics approach has to be reviewed carefully in accordance with the newly established methodology of HMWB. Necessary projections of both approaches have to be executed and some methodologies may have to be modified afterwards.



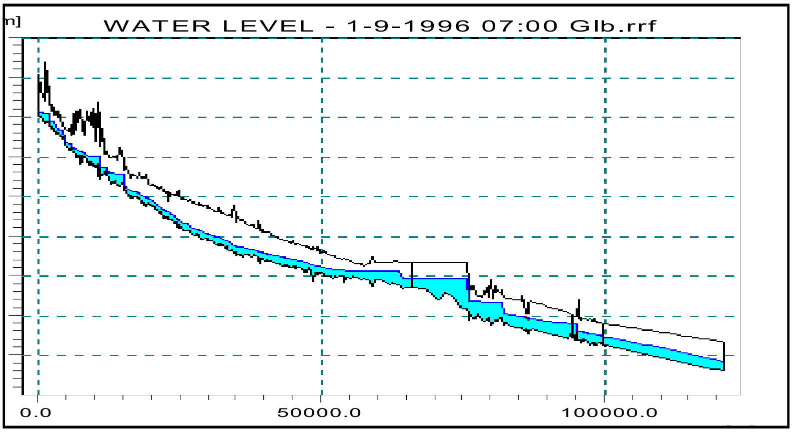


Figure 1B. Master plan of urban stream – outputs of a mathematical model

In the forthcoming projects, tools/methods have to be established for:

- Identification of significant anthropogenic pressures,
- Assessment of the identified pressures on the river status and its sustainability,
- Assessment of the GWB and definition of the risk of failing the FWD recommendations,
- Dissemination of the results through media and capacity building tools, such

as workshops, public meeting, etc.

The objective of a project currently executed by the authors is to apply provisions of the WFD with regard to the identification of heavily modified water bodies and establish a proposal for reference conditions within the basins and sub-basins. The project has to answer, through careful preparation and application of the agreed upon methods, how the designation of HWMB may be done practically, what has been prepared and is already available at the River Basin Authority (monitoring data, analytical tools, status assessment, etc.) and how the reference conditions for those modified bodies may be established. This outcome of the project will help clarify the practicalities of the WFD application in the Czech Republic in the near future.

The results will be disseminated at the workshops and through the means of IT technology. The proposed methodology and a proposal for improvement of weak processes will be delivered to the users. The project will be a milestone for Water Management Plans within the WFD. There is a clear intention of the promoter of this approach to develop practical implementation of the HMWB activities in the selected basins including the fully urbanised ones.

Authors would like to stress some of the conflicts, which apply to UDMP. The team will try to apply the existing data and set-up mathematical models to obtain performance indicators as a basis for further evaluation of water bodies. The definition of HMWB and AWB will be included as a given fact in water management planning. The Regional government has to set up the priorities in the investment policy in accordance with the program measures suggested by the Water Basin Authorities.

The basic definition and designation tests will be conducted on some parts of the rivers which are selected as a pilot domain in the Elbe River Basin. The entire test will be based on the results obtained from simulations, field investigations, data collection and processing, and water quality investigations. The resulting report will contain a methodology applicable in other basins and not only in the Czech Republic. This pilot study will complete the existing list of 35 studies, which have been finalised in the present period. Such a HMWB study will be also very useful in urban areas for nature restoration and rehabilitation of water bodies. Structural funds and cohesion funds of EU may be applied to restoration activities in the regions falling behind the average level of the living standard measured in GDP. The results of implementation of the water body designation process may influence today's practice of MPUS and the authors wish to share their initial opinions on, and experiences with, this process.

2. Water Framework Directive

The Water Framework Directive allows Member States to identify relevant water bodies as "heavily modified" or "artificial" (man-made). The objective set for these bodies is good ecological potential, rather than good surface water status. The relevant provisions are set out in Article 4 of the Water Framework Directive. In particular Article 4.1.a.iii) sets out objectives for "heavily modified and artificial water bodies":

WFD article 4.1

Member States shall protect and enhance all artificial and heavily modified bodies of water, with the aim of achieving good ecological potential and good surface water chemical status at the latest 15 years from the date of entry into force of this Directive, in accordance with the provisions laid down in Annex V, subject to the application of extensions determined in accordance with paragraph 4 and to the application of paragraphs 5, 6 and 7 and without prejudice to paragraph 8;

These provisions mean that Member States must undertake the following tasks:

1. Designation of heavily modified and artificial water bodies according to the definitions in the Directive,
2. Specification of good ecological potential for all of these water bodies.

3. Designation of heavily modified and artificial water bodies

The relevant definitions are provided in Article 2 of the Directive. Criteria for designation of heavily modified bodies are provided in Article 4.3 of the Directive.

3.1 ARTIFICIAL WATER BODIES

Water framework directives define in Article 2.8 the term artificial water body:

"Artificial water body" means a body of surface water created by human activity.

One can summarise the policy from the Guidance issued to the WFD. The definition states:

Artificial Water Bodies are surface water bodies which have been created in a location where no water body existed before and which has not been created by the direct physical alteration, movement or realignment of an existing water body. The application of this guidance is quite clear. Take the following example by Hunt [12]:

- A dam is built across a river. Upstream of the dam, a large body of standing water is created.
- Question: is the large body of standing water to be designated as "artificial" or "heavily modified"?

Answer: The E.U. guidance states that the large body of standing water should be considered heavily modified on the basis that there was a water body there before. This would mean that in order to designate this body of standing water as "heavily modified", the Member State would have to provide justification for the continued existence of the dam in accordance with Article 4.3.

3.2 HEAVILY MODIFIED WATER BODIES

"Heavily modified water body" means a body of surface water which as a result of physical alterations by human activity is substantially changed in character, as designated by the Member State in accordance with the provisions of Annex II. The definition for "heavily modified water body" is

stated in the Article 2.9.

Although this definition refers to Annex II, the criteria for designation are set out in Article 4.3 of the Directive.

Article 4.3 states:

Member States may designate a body of surface water as artificial or heavily modified, when:

(a) the changes to the hydromorphological characteristics of that body which would be necessary for achieving good ecological status would have significant adverse effects on:

- *(i) the wider environment;*
- *(ii) navigation, including port facilities, or recreation;*
- *(iii) activities for the purposes of which water is stored, such as drinking water supply, power generation or irrigation;*
- *(iv) water regulation, flood protection, land drainage, or*
- *(v) other equally important sustainable human development activities;*

(b) The beneficial objectives served by the artificial or modified characteristics of the water body cannot, for reasons of technical feasibility or disproportionate costs, reasonably be achieved by other means, which are a significantly better environmental option.

Such designation and the reasons for it shall be specifically mentioned in the river basin management plans required under Article 13 and reviewed every six years.

3.3 CHARACTERISATION

In the process of characterising water bodies (pursuant to Article 5 and Annex II), Member States must apply specific provisions to heavily modified or artificial water bodies:

1.1.i of Annex II states

For artificial and heavily modified surface water bodies the differentiation shall be undertaken in accordance with the descriptors for whichever of the surface water categories most closely resembles the heavily modified or artificial water body concerned;

1.3.ii of Annex II states

In applying the procedures set out in this section to heavily modified or artificial surface water bodies references to high ecological status shall be construed as references to maximum ecological potential as defined in table 1.2.5. of Annex V. The values for maximum ecological potential for a water body shall be reviewed every six years.

1.4 of Annex II (Identification of Pressures) includes:

Identification of significant morphological alterations to water bodies.

3.4 REPORTING

Section 1.4.2 of Annex V of the Directive also contains specific provisions concerning the presentation of the results of the assessment of ecological potential.

For heavily modified and artificial water bodies, the ecological status classification for the body of water shall be represented by the lower of the values for the biological and physico-chemical monitoring results for the relevant quality elements classified in accordance with the first column of the table set out below. Member States shall provide a map for each river basin district illustrating the classification of the ecological potential for each body of water, colour-coded, in respect of artificial water bodies in accordance with the second column of the table set out below (Table 1), and in respect of heavily modified water bodies the third column of that table:

Table 1. Colour coding for water body classification

Ecological potential classification	**Colour code**	
	Artificial water bodies	**Heavily modified**
Good and above	Equal green and light grey stripes	Equal green and dark grey stripes
Moderate	Equal yellow and light grey stripes	Equal yellow and dark grey stripes
Poor	Equal orange and light grey stripes	Equal orange and dark grey stripes
Bad	Equal red and light grey stripes	Equal red and dark grey stripes

No specific provisions are contained within the Directive concerning the reporting of heavily modified designations. However, the requirements of Annex VII of the Directive require statements concerning the objectives set for water bodies (Item 5).

3.5 GOOD ECOLOGICAL POTENTIAL

The quality elements, which must be applied to Artificial and heavily modified surface water bodies, are set as explained in Section 1.1.5 of Annex V.

The quality elements applicable to artificial and heavily modified surface water bodies shall be those applicable to whichever of the four natural surface water categories above most closely resembles the heavily modified or artificial water body concerned.

4. Pilot project on the Elbe River Basin

To develop a common understanding of the most effective approach to the identification of significant anthropogenic stresses on a river basin and the analysis of a potential impact of such stresses, and to fulfil one of the basic requirements of WFD (Annex II), it is necessary to identify the appropriate tools (such as models, DSS, databases, monitoring networks) needed to carry out the necessary analysis and to identify where the new tools have to be developed or the current ones improved.

The project has to establish the tools/methods for:

- Identification of significant anthropogenic pressures,
- Assessment of the identified pressures on the river status and its sustainability,
- Assessment of the GWB and definition of the risk of failing the FWD recommendations, and
- Dissemination of the results through media and capacity building tools such as, for example, workshops.

The objective of the project is to apply the provisions of the WFD with regard to the identification of heavily modified water bodies and establishing a proposal for reference conditions within the basins and sub-basins. The project has to answer, through careful preparation and application of the agreed methods, how the designation of HWMB may be done practically, what data and tools are already available from the River Basin Authority (monitoring data, analytical tools, status assessment etc.) and how the reference conditions for those modified bodies may be established. The results will be disseminated by the workshops and through IT technology means. The proposed methodology and proposal for improvement of the weak processes will be delivered to the end-users. There is a clear intention of the promoter to work out practical implementation of HMWB activities in the selected basin (Fig. 2).

Figure 2A. Examples of water courses in the pilot area – The Orlice River

Figure 2A. Examples of water courses in the pilot area - The Labe River – the central canalised river reach

4.1 END-USERS OF THE PROJECT RESULTS

The Czech Republic is an accessing country (i.e., in late 2003) and will have to implement the EU WFD in the coming years. The Ministry of Agriculture (MOA) has been appointed as the supervising ministry for the Water Sector, having the responsibility for the Water Framework Directives implementation in the CR. Because a Twinning project is executed in the Elbe basin, focused on the Accession period in the Water sector, the same basin was selected as a test area for implementation of the HMWB methodology. The Elbe River Basin Authority supervises 322 streams of the total length of 4,090 km in the Elbe basin in the Czech Republic. The Elbe RBA has a basic supervision of 231 weirs, 21 dams and 6 ponds in the area of about 15,000 km^2. The main river is the Elbe River itself, which is used for navigation in the lower reach from Prelouc to the border with Germany, at Hrensko (the waterway is 211 km long, with 30 lock chambers). The river has been fully canalised in this reach and the former river channel is fully modified. Several tributaries were also modified in the past, however, their revitalisation might be an interesting alternative in the near future. The domain of interest complies with the conditions for implementing the HMWB methodology in the Czech Republic.

5. The target group

Based on the Competency Act No. 122/1997 and Water Law Decree NO. 254/2001, the Ministry of Agriculture (MOA) became the central state authority for water management including maintenance of water courses, control of land-water regime and the use of water. For this reason the MOA represents the primary direct end-user, because the MOA is fully responsible for implementation of all activities related to

WFD. The Ministry of Environment (MOE) remains the central state authority for protection of natural water balances and protection of surface and groundwater quality. Designation of HMWB and a detailed classification of the stress and ecological status of the river/stream reaches will essentially assist MOE in prioritising their environmental protection policy. River Basin Authorities (Elbe, Vltava, Ohre, Odra and Morava) will benefit from the results of the project by gaining experience with implementation of a totally new activity of designation of HMWB in accordance with the methodology modified for the conditions in the Czech Republic. The direct end-users will be indeed the Elbe River Basin Authority as a partner in the project; however, the managers of the lower order streams in the basin will be introduced to this activity through information events and other means.

6. Conclusions

A comprehensive and general Decision Support System (DSS) promoting the harmonisation of the environmental standards in the Czech Republic with the EU legislation has been established. The DSS combines several activities and components. The HMWB designation plays a very important role in water-management planning and investment policy of the newly established regions. The economic and financial implications of these strategies have been analysed both for optimum and accelerated time frames to achieve compliance with the EU regulation. This is fully valid for the HMWB designation. It is expected that the designated water bodies will become later a regular part of the DSS tool. The Decision Support System has been designed to model pre-defined scenarios and their economic and technical conditions. Relevant data needed for the application of different models and for the evaluation of specific scenarios have been included in a database comprising a combined GIS (ARC VIEW) and a database product (WINbase). The introduction to DSS and implementation of DSS at the regional level for evaluation of best strategies for the programmes recommended by the River Basin Authority and further planning within water management must be completed for the designated water bodies. This water body definition will create the backbone platform for the DSS from the point of view of the river management sector.

A team of Czech and foreign experts has been heavily involved in the development of the system, which is now being implemented in the Czech Republic for river basin planning. The tools, which are used or developed for the project, are generic and applicable also to other river basins, when the pilot study is completed.

The basic components of the DSS are shown in Figure 3. The DSS facilitates access to relevant information on the national scale and provides a computational capability for the analysis and evaluation of different options to assist in the identification of viable strategies.

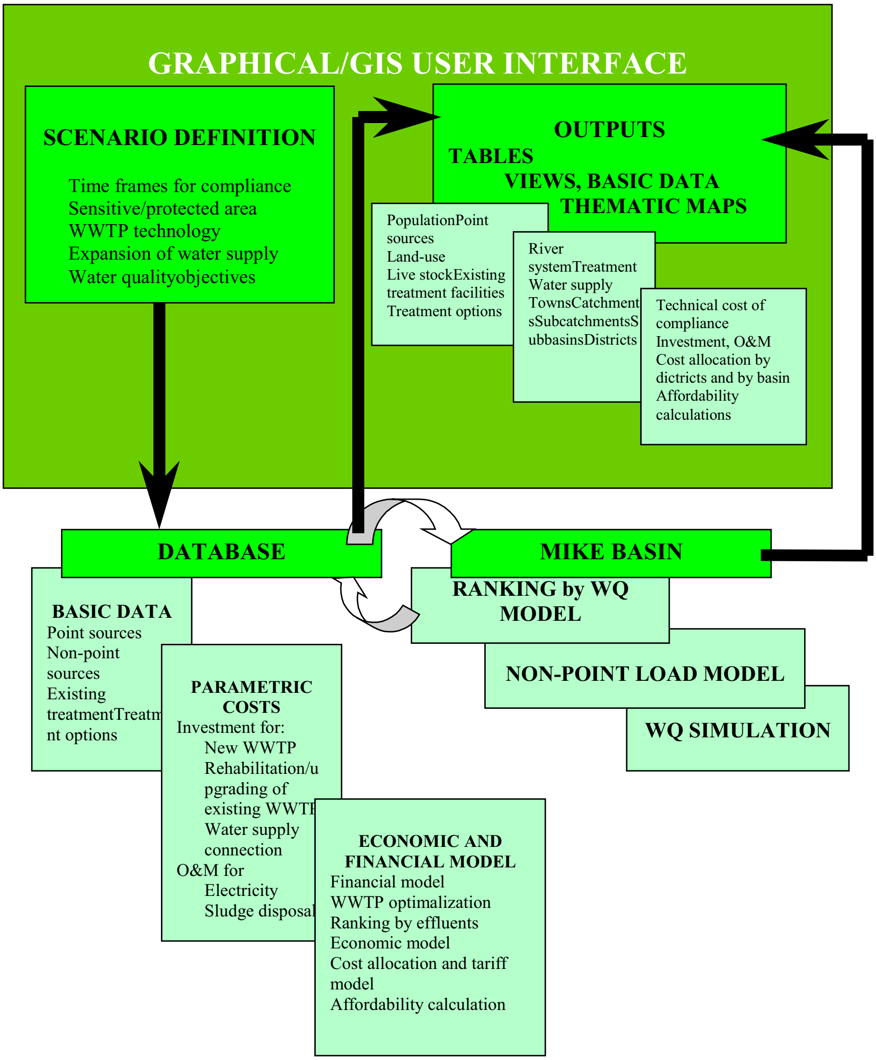


Figure 3. Decision support system (DSS) schematic

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THE ZURICH STREAM DAY-LIGHTING PROGRAM

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1. Introduction

In the concept of separating clear extraneous water from the sewer system, the Zurich day-lighting program plays an important role. The objective of avoiding diversion of stream water into the sewers led to the idea of reopening and revitalising brooks and streams and using them as a part of the separate clean water system [1]. This concept provides a solution to the problem faced by many communities, as the traditional way of coping with stream water and wastewater was to amalgamate them into the sewer system. As towns grow, the disposal of wastewater becomes a larger problem and the treatment of sewage requires economic considerations. When all the extraneous water like stream water, infiltration water from house drains, water from fountains and cooling devices is fed into the sewage system, there will be problems. The obvious solution is to separate the clean water at the source, so that the sewage treatment plants have to process only sewage. Thus, reopening (day-lighting) streams as a part of the separate system for clean water is an ideal solution from the economic, ecological and aesthetic points of view. The Zurich stream day-lighting concept also encompasses the streams, which have been drained into sewer pipes. Historically, this was the way to deal with small streams in urban areas. The day-lighting of brooks and streams is also an attractive alternative enhancing the urban landscape.

2. Background

The City of Zurich is surrounded by hills with numerous springs that form over fifty smaller or larger streams flowing into the city. During the last 130 years of the city development and construction, about 100 km of these previously open waters disappeared from the surface. Road construction, risk of flooding and pollution of these brooks prior to the construction of area-wide sewerage systems were the main reasons why these water flows were channelled into large underground pipes, which eventually became sewers and were connected to the sewage treatment plants.

As a result, the city's two wastewater treatment plants had to deal with large quantities of "clean" runoff, which increased operational costs and diminished the efficiency of the wastewater treatment process. An additional consequence was the deterioration of the urban landscape, loss of valuable public recreation zones, and degradation of the natural habitat of plants and animals.

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As this was a nation-wide problem, the revision of the Swiss water protection law in 1991 stipulated that clean runoff and unpolluted rainwater should seep directly into the ground or, where this is not possible, should be diverted into a stormwater drainage system which is separated from the pipes carrying the wastewater.

3. Extraneous Water in the Sewer System

Historically, the Zurich sewer system is built mostly as a combined one. As a consequence of the introduction of stream water into this sewer system, about one third of the flow was clean extraneous water. Because of this, Zurich started to modify the combined sewer system to a partially separate system.

The existing combined sewers serve as carriers of polluted water; that means for conveying wastewater from households and industries and for rainwater runoff from streets with much traffic and from other polluted areas. A new system serves for the diversion of all the mentioned extraneous clean water. This clean water system has to be well planned in order to achieve the best results in the most cost effective manner. Since a considerable part of the extraneous water in Zurich's sewer system originates from many streams, the separation of this water promises to be the most successful solution. Even if the streams are small, the relatively continuous flow throughout the year adds up to a considerable volume of water. For this reason it is planned to divert all streams, except the least important ones, directly to the new system, which drains directly to the receiving water body (Fig. 1). Where possible, clean water from springs and fountains, and yard drainage and cooling waters will be connected to this system.

As the space in the urban environment is limited, this clean water system is generally designed for not more than two to five times the dry weather flow, so the dimensions are not very large. When it rains, the surplus overflows to the sanitary sewers (formerly combined sewers), which normally have sufficient capacity.

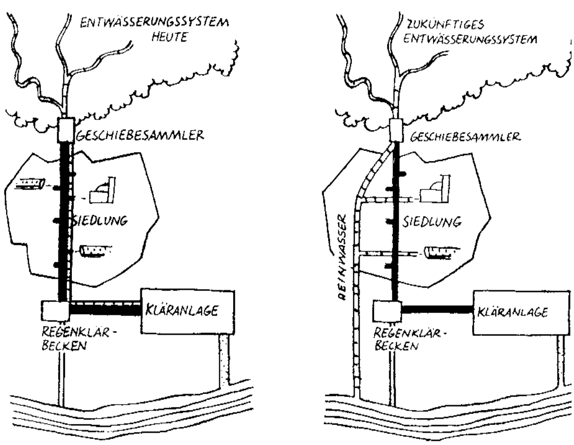

Fig. 1. The new drainage system: clean waters, which previously flowed to the WWTP, are diverted directly to the receiving water

At the start of this programme, over fifteen years ago, the idea that the urban drainage systems must be built with pipes was still common. But there is no reason to do it in every case. A stream can also drain clean waters.

4. Streams not pipes

To build a stream channel instead of a pipe is a feasible solution in urbanised areas. For this reason, in the context of the Zurich urban drainage master planning, the so-called "concept of streams" has been worked out. After having considered the need for space, legal and technical aspects and other arguments, the evaluation showed that over 30 km of brooks could be constructed in the City of Zurich (Fig. 2).

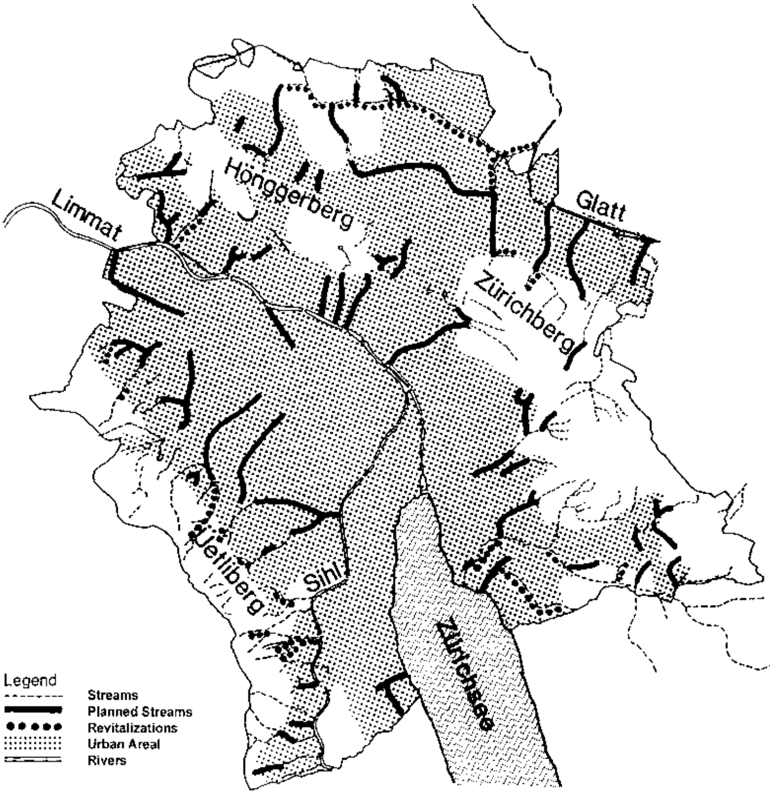

Fig. 2. The Zurich "Stream day-lighting Program" shows the opportunities to open streams

5. Objectives

A stream is a natural habitat for many plants and animals. The structure of the bed and banks allows bacteria, insects, fish and many other animals to develop and live.

A large spectrum of plants grow along a brook. The more natural the water body is, including its bed, banks and riparian area, the more animals and plants will live there.

The Zurich "stream day-lighting concept" is a typical result of integral thinking. The program was approved by the Zurich city government in 1988, and aimed to achieve the following objectives:

1) to improve the recreational qualities of urban neighbourhoods and thus make them more attractive,
2) to restore lost habitat for plants, insects and small animals, enhancing the relationship between city residents and their natural environment, and
3) to reduce the amount of clean water flowing through the wastewater treatment plants and thus improve the quality and the efficiency of treatment process.

6. Results

During the last 15 years, in more than 40 projects, 16,000 m of streams were day-lighted or revitalised in the City of Zurich [2].

- The day-lighted streams represent in many ways the wishes and the needs of the population for “more nature” in the city.
- The streams are especially popular with children, who often use them as playgrounds.
- Brooks are an important factor in landscape and urban residential planning.
- To date, of the estimated 800 l/s total extraneous water in the sewer system, approximately 300 l/s have been diverted to the new streams. In this way the sewage treatment plant is less loaded.

7. Examples of reopened and revitalised streams

7.1 SAEGERTENBACH

Saegertenbach Creek is fed by a little brook, which originates in the woods. It has been opened to divert the stormwater from a residential area, especially from roofs, and forms now a part of the landscaping (see Figs. 3 and 4).

The Döltschibach is one of various streams originating from "Uetliberg", which is a hilly area surrounding the western part of the City of Zurich. Shortly after water leaves the woods, it disappears in the underground and the stream water was diverted together with wastewater to the wastewater treatment plant. The day-lighted stream has a capacity of 200 l/s. During storms, if the flow is higher, the overflow is diverted into sewers as before (Figs. 5 and 6).

Fig. 3. Before opening Saegertenbach

Fig. 4. Saegertenbach Creek as a part of landscaping in the residential area of Döltschibach

Fig. 5. Along the "in der Ey"- Street, despite the narrow space, it was possible to build the day-lighted stream between the sidewalks and the private properties.

Fig. 6. Further downstream, stream flows through a residential area

7.2 ALBISRIEDER DORFBACH

The Albisrieder Dorfbach brook was one of the first big projects and became day-lighted over a length of 2,500 metres between 1989 and 1991. The stream flows through public and private properties. Its capacity is up to 200 litres per second, the intermediate flow ranges from 10 to 20 litres per second (Figs. 7 and 8).

Fig. 7. The view before opening Albisrieder Dorfbach in a private residential area.

Fig. 8. The view after day-lighting the stream.

7.3 BINZMUEHLEBACH

The Binzmuehlebach brook is 800 metres long and its average flow is about 15 litres per second. The water originates from a catchment area of 200 hectares of woods. Additionally it gets the rainwater and infiltration waters from the nearby residential area (Fig. 9).

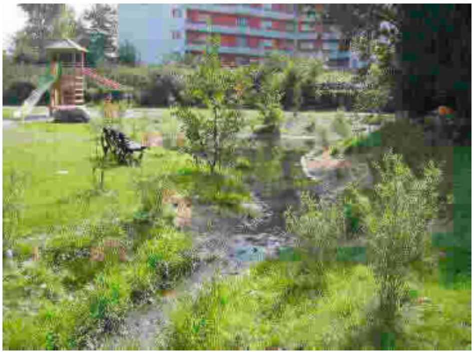

Fig. 9. The Binzmuehlebach was reopened in 1988 – 2000 as a part of conversion of a former industrial area to a residential area.

7.4 FRIESENBERGBACH

A part of the Friesenberg stream flows through a new residential area. The stream renaturalisation was planned and built together with houses in 1991. The maximum flow is 100 litres per second; the excess flow is diverted into the sewer system (Figs. 10 and 11).

Fig. 10. The Friesenbergbach brook became a part of the landscape of the new residential area.

Fig. 11. Small new lake, fed by the Friesenberg stream, is used for recreation, including swimming.

7.5 MANEGGBACH

In 1998 the Maneggbach has been opened over a length of 100 metres. The dry weather flow is only 5 litres per second, during storms the flow rises up to 800 l/s.

Fig. 12. The day-lighted stream has been integrated into the landscaping of a new residential area.

7.6 NEBELBACH

The Nebelbach project was built in 1991. It represents an example of a day-lighted stream located in a public street. It diverts the dry weather flow of up to 70 litres per second. There is trout living in this stream. The excess water is diverted into a storm sewer.

Fig. 13. The day-lighted Nebelbach brook forms part of the public street.

7.7 WOLFGRIMBACH

The Wolfgrimbach brook has been day-lighted in 1998, over a length of 800 metres. The day-lighting had to be done in a very narrow space. It is designed to divert up to 100 l/s. The active participation of the neighbourhood was very important and helped to finance the project.

Fig. 14. Wolfgrimbach reach in the garden of a private property.

8. The sustainability of the Stream Day-Lighting Program

The sustainability of the Zurich program has been examined using the six sustainability criteria proposed by Hugentobler and Brandli-Stroh [3]. The Zurich day-lighting program was found to be sustainable at all six levels.

With respect to the *Cultural System*, the relationship between humans and nature in urban neighbourhoods and the understanding of, and care in dealing with, nature are important positive factors, which are strengthened by the open streams. The *Social Systems* include increased social interaction and networking in the neighbourhoods, citizen participation in the planning and implementation process and interdisciplinary cooperation across the city government departments. Many stream opening projects have contributed to strengthening the social systems and in turn have been improved by citizen participation.

The *Human (individual) Systems* regeneration of streams in the immediate living environment offers aesthetic amenities and sensory stimulation, and the opportunities (for children) to play in, experiment with, and creatively use the immediate living environment.

In the *Biological System* the increased diversity of plants and animals in urban neighbourhoods and more linkages between natural habitats contribute to sustainability.

In the *Chemical/Physical Systems*, energy savings in the wastewater treatment process and reduction of the extent of sealed urban lands are very positive. Risk of flooding of urban neighbourhoods during heavy rainfall and the risk of water pollution with toxic substances/chemicals are the only factors, moving slightly away from sustainability.

9. Success factors

Many factors helped make the Zurich Stream day-lighting Program very successful. It is important to involve all decision making stakeholders in the process. The approval of the stream day-lighting program in an early phase by the city government showed the willingness of politicians to support new ideas.

The formation of the "stream day-lighting group", an interdepartmental task force, helped to co-ordinate different activities in this field between the city departments and to overcome the resistance within the city administration.

The use of the planned revision (at that time) of the Swiss water protection law, which prescribes the separation of clear water runoff from the sewer system, helped in the argumentation for the stream day-lighting.

From the point of view of the integrated water management, the location of the program in the sewer department, with experience in project management and financial matters, helped in the implementation.

The development of specific guidelines and preparation of a list and plan of possible streams to be day-lighted or revitalised had helped the departments to recognise where actions needed to be taken and to include stream projects in their activities.

Guidelines for the technical design and the implementation of the stream opening projects help maintain a uniform philosophy. With information disseminated through local press, holding meetings in the neighbourhoods, discussions with the affected property owners and the involvement of school classes helped to convince the public.

In May 2003, the City of Zurich received the water price for the Stream day-lighting Program comprising many opened and revitalised streams in the urban environment.

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ADVANCES IN MODELLING THE EXPERIMENTAL DATA ON POLLUTANT DYNAMICS IN EPHEMERAL STREAMS INFLUENCED BY TRANSIENT STORAGE IN DEAD ZONES

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1. Introduction

Many recent studies showed that most of the pollution loads discharged to receiving waters by sewer overflows during the year are caused by few dozen rainfalls. Therefore it is very interesting to evaluate the influence of this intense contamination on water quality, with time scales limited from minutes to few hours, particularly in ephemeral streams, where such conditions are critical for the aquatic ecosystem.

In some countries these observations encouraged the definition of water-quality standards to be complied with during overflow events, such as the so-called FIS (Fundamental Intermittent Standards) established in the English UPM (Urban Pollution Management) Procedure [1]. Such procedure is focused on the persistence of the pollutant concentrations under or above given threshold values for just few indicators, like dissolved oxygen (DO) and ammoniac nitrogen ($N\text{-}NH_3$), while their peak values are considered less important; the return period of the persistence time is assumed to be related to the feasibility of maintaining specific kinds of aquatic life. Such approaches can not be implemented directly in other situations without a sufficient knowledge of both experimental data and proper models, especially in climatic conditions different from the UK ones.

Therefore, several models have been developed to simulate the dynamics of pollutants in natural streams. However, they do not always seem to be able to give reliable information on pollution propagation in conditions which are different from those of their calibration.

Most of the existing one-dimensional models (i.e., QUAL2E [2], ISIS [3], WASP [4] and MIKE 11 [5]) solve the so-called Advection Dispersion Equation (ADE) [6] reported below, neglecting the strong influence that transient storage (Fig. 1) can have on the pollutant dynamics. To highlight the importance of dead zones, some different models are hereby analysed in a simple case representing a temporally limited discharge event under the following hypotheses: constant discharge in the receiving stream, without lateral inflow or outflow in the reach (steady flow); suspended load only (negligible effects of reach bed sedimentation and resuspension); complete vertical and lateral mixing, which indeed occurs only after a certain minimum distance from the input [7, 8]; and, slug injection of conservative or

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non-conservative solute (Fig. 2).

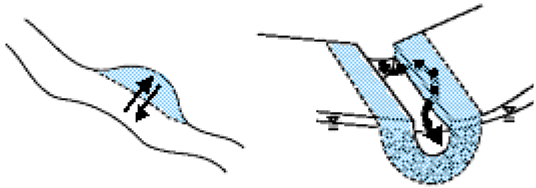


Fig. 1 - Transient storage in dead zones that are pools and streambed porous media (arrows denote solute exchange between main channel and dead zones).

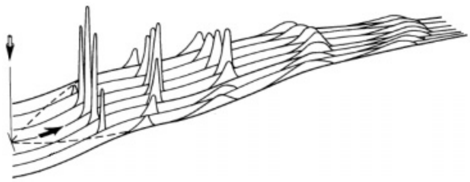


Fig. 2 - Lateral mixing and longitudinal dispersion patterns and changes in distribution of concentration downstream from a spill of mass M (single centred slug injection of tracer).

In a stream with homogeneous hydraulic and geometric properties, the mass balance in a control volume, in an Eulerian approach, yields the above mentioned ADE:

$$\frac{\partial C}{\partial t} = U \cdot \frac{\partial C}{\partial x} + D \cdot \frac{\partial^2 C}{\partial x^2} \tag{1}$$

the solution of which, for the particular boundary condition $C(0,t) = M \cdot \delta(t)$, where $\delta(t)$ is the Dirac's delta function, can be expressed by:

$$C(x,t) = \frac{M}{2A \cdot \sqrt{\pi \cdot D \cdot t}} e^{-\frac{(x - U \cdot t)^2}{4D \cdot t}} \tag{2}$$

where $C(x,t)$ is the average concentration in the cross section [M/L^3], M is the mass spilled at $x = 0$ [M], U is the average stream velocity [M/T], A is the average cross-sectional area (with $U = Q/A$) [L^2] and D is the dispersion coefficient [L^2/T]. The same expression can be obtained, according to a Lagrangian approach, also with a Gaussian "random walk method" [9].

A different random walk method based on the probability density function of the Shifted Gamma Distribution [10] seems to work better, because its exponential term can take into account the delay due to the presence of dead zones. If M is the mass spilled and Q is the discharge conveyed in the reach, the concentration is:

$$C(t) = \frac{M}{Q} \cdot \frac{\alpha^{\gamma} \cdot (t - t_0)^{\gamma - 1} \cdot e^{-\alpha \cdot (t - t_0)}}{\Gamma(\gamma)} \tag{3}$$

where:

$$\Gamma(\gamma) = \int_0^{\infty} e^{-\beta} \cdot \beta^{\gamma - 1} \cdot d\beta \tag{4}$$

A good fit of the experimental data has been achieved with the OTIS (One-dimensional Transport with Inflow and Storage) model [11, 12], compiled by USGS, which considers explicitly the dead zones. This software solves, using finite differences, a coupled set of partial differential equations derived from a mass balance in the two interactive control volumes, in which it divides each reach (the main channel and the storage zone, as shown in Fig. 3):

$$\begin{cases} \dfrac{\partial C}{\partial t} = -\dfrac{Q}{A} \cdot \dfrac{\partial C}{\partial x} + D \cdot \dfrac{\partial^2 C}{\partial x^2} + \alpha \cdot (C_S - C) \\ \dfrac{dC_S}{dt} = \alpha \cdot \dfrac{A}{As} \cdot (C - C_S) \end{cases} \tag{5}$$

where C is the solute concentration in the main channel, C_S is the solute concentration in the storage zone, A is the main channel cross-sectional area, As is the storage zone cross-sectional area, D is the dispersion coefficient and α $[T^{-1}]$ is the storage zone exchange coefficient. The first equation is the ADE with the last term in addition, while the second is the continuity equation applied to the dead zones: both of them assume that the direction of the mass exchange process depends on the sign of $(C - C_S)$, that is the concentration difference between the main channel and the storage zone.

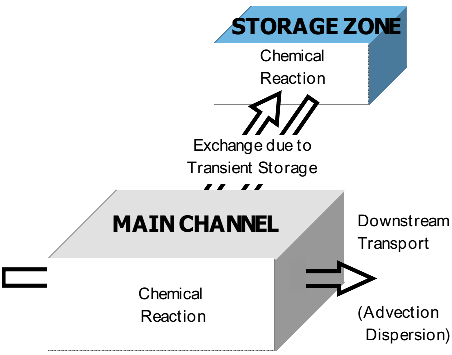


Fig. 3 – Conceptual base of the OTIS model that includes the two interactive control volumes, the main channel and the storage zone.

In this paper the main one-dimensional models reported in the literature are analysed and compared with the field data of "time of travel" (ToT) collected in an experimental survey carried out on the Lura, a small stream in an urban area near Milan. It has been chosen because it is an ephemeral stream that flows through many towns and receives high loads from CSOs coming from the sewer systems serving about 350,000 population equivalents (both municipal and industrial). The study of three different kinds of reaches shows that the influence of intense and temporally limited loads on watercourses with low flows depends on their different morphology. To enhance the fit of field data, without the algorithms implemented by OTIS, two original conceptual models, based on a hydrological analogy, are hereby proposed. At first they have been developed to simulate the transport of a conservative pollutant (as the tracer used in experiments was); afterwards a first order decay rate has been included in those two models.

2. Experimental reaches

Three experimental reaches, shown in Fig. 4, have been selected on the Lura stream, which has a total length of about 40 km and is representative of many streams around Milan [13].

The aim is to screen the behaviour of different stream morphologies: a straight reach N-M, a winding reach D-G, and a highly meandering reach S-I-II. In particular, the reach S-I-II includes an intermediate station (I) that allows collecting additional data to compare the performance of the models when simulating the propagation along reach S-II as a whole, rather than as the results of the propagation along the two sub-reaches in series S-I and I-II.

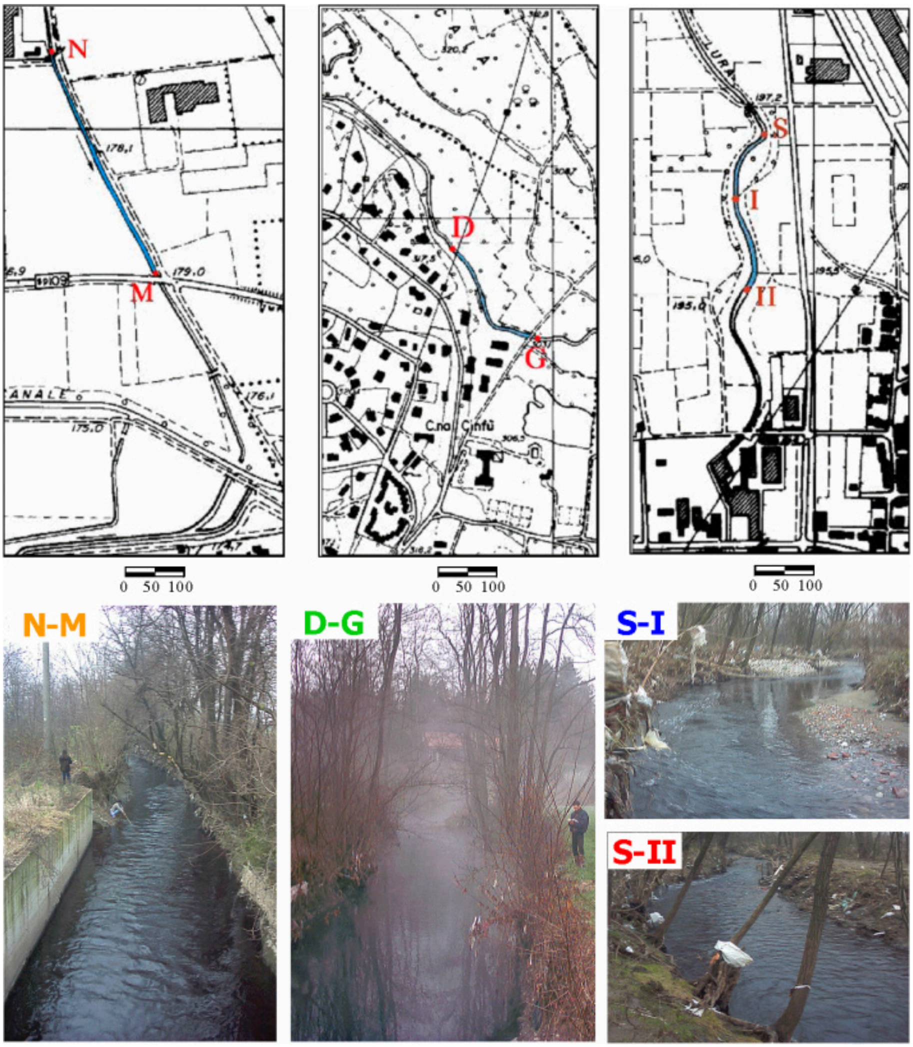


Fig. 4. The three experimental reaches on the Lura stream, characterised by different morphologies: straight in N-M, winding in D-G, and highly meandering in S-I-II.

At those sites, 35 ToT surveys have been carried out in dry weather with steady flow, each time injecting a known mass of a conservative tracer (salt) in the upstream section and measuring the consequent increase of conductivity in the downstream section with respect to its base value [9].

Unfortunately, 18 of these 35 ToT surveys had to be discarded because of gauge problems, discharge varying too quickly or unacceptable conductivity sensor noise.

Among the remaining 17 runs summarised in Tab. 1, one refers to reach N-M, five to reach D-G, four to reach S-I and, seven to reach S-II. Furthermore, 6 discharge measurements were collected by means of current meters in section G and 1 in section S to verify (in all cases successfully), the discharge values given by the corresponding ToT surveys [13].

DATE [yyyy/mm/dd]	REACH	LENGTH x [m]	DISCHARGE Q [m^3/s]	INPUT MASS M [kg]	ToT [s]	VELOCITY $\overline{U}=\frac{x}{ToT}\left[\frac{m}{s}\right]$	AVERAGE AREA $\overline{A}=\frac{Q}{\overline{U}}\,[m^2]$
2002/07/03	S-I	132	3.11	6	162	0.81	3.82
2002/07/10	S-I	132	0.25	6	631	0.21	1.20
2002/07/17	S-I	132	0.81	4	327	0.40	2.01
2002/07/25	S-I	132	0.22	3	833	0.16	1.39
2002/07/03	S-II	290	3.11	6	288	1.01	3.09
2002/07/10	S-II	290	0.25	6	1220	0.24	1.05
2002/07/17	S-II	290	0.81	4	564	0.51	1.58
2002/07/25	S-II	290	0.22	4	1340	0.22	1.02
2002/10/03	S-II	290	0.35	3	918	0.32	1.11
2002/11/06	S-II	290	0.27	3	1113	0.26	1.04
2002/12/12	S-II	290	1.22	3	408	0.71	1.72
2002/10/17	D-G	248	0.47	3	960	0.26	1.82
2002/10/24	D-G	248	0.55	3	862	0.29	1.91
2002/11/06	D-G	248	0.40	3	1107	0.22	1.79
2002/12/12	D-G	248	1.23	3	516	0.48	2.57
2002/12/19	D-G	248	0.98	3	610	0.41	2.41
2002/07/17	N-M	406	1.83	4	433	0.94	1.95

Table 1. Main data from the 17 valid ToT experimental surveys.

3. Proposed conceptual models for conservative pollutants

The basic concept of the "Instantaneous Unit Pollutograph" (IUP) approach for modelling the dynamics of pollutants in stream reaches [14, 15] is inspired by the "Instantaneous Unit Hydrograph" (IUH) approach used in hydrology [16], just replacing, in the convolution integral process, rainfall inflow $P(\tau)$ with concentrations at the upstream section $C_0(\tau)$ at time τ, $IUH(t-\tau)$ with $IUP(t-\tau)$ as time-lag transformation of a unit input impulse, and finally discharge $Q(t)$ with output concentrations at the downstream section $C(t)$ at time t:

$$Q(t)=\int_0^t P(\tau)\cdot IUH(t-\tau)\cdot d\tau \quad (6a) \quad \Longrightarrow \quad C(t)=\int_0^t C_0(\tau)\cdot IUP(t-\tau)\cdot d\tau \quad (6b)$$

This approach is not new, since it has been already applied to the simulation of the build-up of soluble pollutants in streams and estuaries [17] and to the transport of solutes in groundwater at a catchment scale [18, 19], but it is here expressly applied to the dissolved transport processes along rivers and streams receiving highly transient pollutographs, and thereby developing the idea of a routing procedure [7, 20, 21].

According to this idea, any time-variable concentration distribution $C_i(\tau)$ at an upstream boundary of a reach (i.e., at the progressive $x=L_i$) can be discretised into a series of input components. Then, each one is propagated downstream to the progressive $x = L_{i+1}$ using a suitable $IUP_{i \to i+1}(t-\tau)$ function, which represents the one-dimensional propagation of a unit input impulse from section i at time τ to section i+1 at time t. The resulting components at time t are added up to obtain the time-variable concentration distribution:

$$C_{i+1}(t) = \sum C_i(\tau) \cdot IUP_{i \to i+1}(t-\tau) \cdot \Delta\tau \tag{7}$$

Afterwards, such resulting concentration time series $C_{i+1}(t)$ at the downstream end of the first computational reach can be seen as a new upstream condition to estimate the propagation from section i+1 to section i+2 by means of a suitable function $IUP_{i+1 \to i+2}$, which can be, in general, different from $IUP_{i \to i+1}$. So the calculation proceeds to the following downstream sections (Fig. 5).

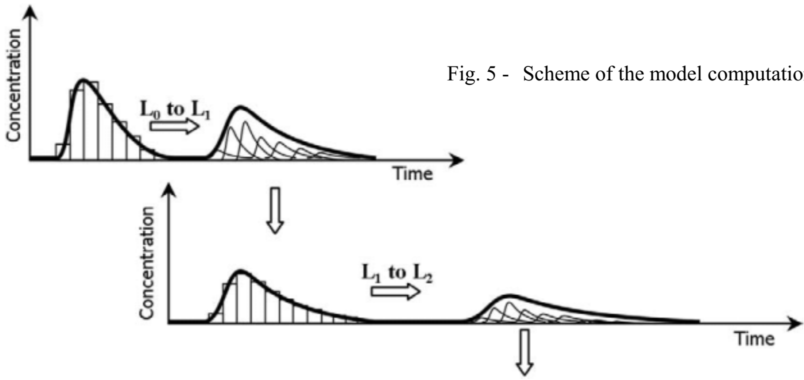


Fig. 5 - Scheme of the model computational procedure.

The first implementation of the above mentioned IUP approach was developed for non-conservative pollutants referring to discretised components of rectangular shape [14, 15], each one propagated by means of the analytical solution of the one-dimensional ADE eq. (1), in the case of single rectangular input [22] valid for t > Δτ:

$$C(x,t) = \frac{C_0}{2}\left\{\exp\left[\frac{U \cdot x}{2D}(1-\Gamma)\right] \cdot \left[\operatorname{erfc}\left(\frac{x - U \cdot t \cdot \Gamma}{2\sqrt{D \cdot t}}\right) - \operatorname{erfc}\left(\frac{x - U \cdot (t-\Delta\tau) \cdot \Gamma}{2\sqrt{D \cdot (t-\Delta\tau)}}\right)\right] + \right.$$

$$\left. + \exp\left[\frac{U \cdot x}{2D}(1+\Gamma)\right] \cdot \left[\operatorname{erfc}\left(\frac{x + U \cdot t \cdot \Gamma}{2\sqrt{D \cdot t}}\right) - \operatorname{erfc}\left(\frac{x + U \cdot (t-\Delta\tau) \cdot \Gamma}{2\sqrt{D \cdot (t-\Delta\tau)}}\right)\right]\right\} \tag{8}$$

being:

$$\operatorname{erfc}(z) = 1 - \operatorname{erf}(z) = 1 - \frac{2}{\sqrt{\pi}} \int_0^z e^{-\beta^2} d\beta \tag{9}$$

where: C(x,t) = concentration at (x, t) [M/L^3], C_0 = concentration of the rectangular input [M/L^3], x = longitudinal distance from input [L], t = time lag from start [T], U = average velocity [L/T], D = average longitudinal dispersion coefficient [L^2/T], Δτ = time base of the rectangular input [T], $\Gamma = (1 + 4\lambda \cdot D / U^2)^{1/2}$, and λ = pollutant first order decay rate [T^{-1}].

In addition to the fact that the structure of eq. (8) is quite complicated, including the complementary error functions erfc, the advantage of its use in the IUP approach is that it allows discretising input pollutographs on the basis of the physical phenomena of advection and dispersion but without taking into account dead zones.

The set up of the new models has been based on the experimental results of the above mentioned ToT survey on the Lura stream, beginning with the hypothesis of conservative solute and later adding a decay function to describe the general case of non-conservative solutes. Again, some basic ideas have been borrowed from hydrology. The first, inspired by the lumped conceptual linear reservoir model for hydrological catchments, is that dead zones behave like a single reservoir with routing constant K [T], giving a delayed release of pollutants with respect to the simple ADE propagation. The second, inspired by the lumped conceptual Nash model for hydrological catchments [23], is an enhancement of the first, as it considers that dead zones behave like N reservoirs in series (where N does not have to be an integer, and all have the same routing constant K), which describes a delayed release of pollutants with respect to the simple ADE propagation. Comparing the ADE propagation in series to a single reservoir or to a series of N reservoirs, the so-called ADEK and ADENK models have been built [13]. Indeed, both of them work for the ADE propagation, and provide a simple ADE solution (2) for instantaneous spill of a known mass M, instead of the complex solution (8) for rectangular input components.

It is important to highlight that this kind of linear reservoir approach is different from the ADZ approach [24], which does not imply any idea of convolution, but discretises the differential solute mass balance in a river reach, considered as an "imperfectly mixed system", where a dissolved solute undergoes pure advection followed by dispersion in a lumped active mixing zone (AMZ).

As the IUH of a single reservoir with routing constant K is:

$$\mathrm{IUH}(t) = \frac{1}{K} \cdot e^{-\frac{t}{K}} \tag{10}$$

the IUP of the ADEK model is given by the superimposition of the ADE solution (2) for instantaneous spill, divided by M and multiplied by Q, with the single reservoir effect:

$$\mathrm{IUP}_{\mathrm{ADEK}}(t) = \int_0^t \frac{Q}{2 \cdot A \cdot \sqrt{\pi \cdot D \cdot \tau}} \cdot e^{-\frac{(x - U \cdot \tau)^2}{4 \cdot D \cdot \tau}} \cdot \frac{1}{K} \cdot e^{-\frac{t-\tau}{K}} \cdot d\tau \tag{11}$$

Furthermore, the IUH of N reservoirs, all with routing constant K, can be written as:

$$\mathrm{IUH}(t) = \frac{1}{K \cdot \Gamma(N)} \cdot \left(\frac{t}{K}\right)^{N-1} \cdot e^{-\frac{t}{K}} \tag{12}$$

where $\Gamma(N) = (N-1)!$, if N is an integer, otherwise:

$$\Gamma(N) = \int_0^\infty e^{-\beta} \cdot \beta^{N-1} \cdot d\beta \tag{13}$$

The IUP of the ADENK model is given by the superimposition of the ADE solution (2), divided by M and multiplied by Q, with the multiple reservoir effect:

$$\mathrm{IUP}_{\mathrm{ADENK}}(t) = \int_0^t \frac{Q}{2 \cdot A \cdot \sqrt{\pi \cdot D \cdot \tau}} \cdot e^{-\frac{(x - U \cdot \tau)^2}{4 \cdot D \cdot \tau}} \cdot \frac{1}{K \cdot \Gamma(N)} \cdot \left(\frac{t-\tau}{K}\right)^{N-1} \cdot e^{-\frac{t-\tau}{K}} \cdot d\tau \tag{14}$$

In the end, any time-variable concentration distribution $C_i(\tau)$ at a section i upstream can propagate to a section i+1 downstream, taking into account advection, dispersion and transient storage in dead zones by means of the discretised convolution of $C_i(\tau)$ together with the IUP_{ADEK} or IUP_{ADENK} expression, thus obtaining:

$$C_{i+1}(t) = \Sigma\, C_i(\tau) \cdot IUP_{i \to i+1}(t-\tau) \cdot \Delta\tau \tag{15}$$

and so on, step by step, for downstream reaches.

Since $Q \cdot C_i(\tau) \cdot \Delta\tau = M_i(\tau)$ by definition, whatever is the adopted form of the IUP expression, such computational procedure has been called ADESS (ADE plus Sum of Spills), because any upstream pollutograph $C_i(\tau)$ is discretised and propagated as a sequence of spills, each one of mass $M_i(\tau)$. In the particular case of one single spill of a known mass M at section i, as in tracer studies, ADESS becomes:

$$C_{i+1}(t) = (M/Q) \cdot IUP_{i \to i+1}(t) \tag{16}$$

4. Comparison of the simulated concentration with field data

All the 17 concentration versus time curves collected in the three different reaches (Innocenti, 2003) have been simulated with all the above mentioned models (ADE, Shifted Gamma Distribution, OTIS, ADEK, ADENK); the values of their parameters have been chosen in order to get the best fit of the field data through different optimisation methods, such as weighted least squares and the method of moments. Due to lack of space, only one example for each reach of the comparison between the various simulations and the observed data is shown in Fig. 6. However, the example represents well all the omitted ones.

The ADE solution, with {A, D} optimised by the weighted least squares method ("weighted" with the term $C^2_{observed,i}$ to increase the importance of the peak value) with:

$$\min_{U,D} \Sigma_i [(C_{observed,i} - C_{ADE,i})^2 \cdot C^2_{observed,i}] \tag{17}$$

seems to be unable to simulate the skewness of the data in all the meandering reaches (S-I-II, D-G) with remarkable dead zones, having a rising branch less steep than the real one and a shorter tail; moreover, it shows a certain underestimation and some delay in the peak value.

The ADE solution, with {A, D} optimised by the method of moments [25] with:

$$\begin{cases} ToT = \dfrac{x}{U} + \dfrac{2D}{U^2} \\ \sigma_t^2 = \left(\dfrac{x}{U} + \dfrac{4D}{U^2}\right) \cdot \dfrac{2D}{U^2} \end{cases} \quad \text{and} \quad \begin{cases} ToT = \dfrac{\int [c(t) \cdot t]\, dt}{\int c(t)\, dt} \\ \sigma_t^2 = \dfrac{\int [c(t) \cdot t^2]\, dt}{\int c(t)\, dt} - ToT^2 \end{cases} \tag{18}$$

where ToT is the time of travel of the solute mass centroid and σ^2_t is the concentration variance in time, seems to work even worse.

On the contrary, in the straight reach N-M, where there is a negligible delay effect of dead zones, also the ADE analytical solution fits the field data well.

The Shifted Gamma Distribution, with its parameters $\{\alpha, \gamma, t_0\}$ optimised by the weighted least squares method:

$$\min_{t_0,\alpha,\gamma} \Sigma_i[(C_{observed,i} - C_{GAMMA,i})^2 \cdot C^2_{observed,i}] \tag{19}$$

seems to have only the peak value a bit lower than the real one, but the shape follows the observed curve.

The OTIS model, however, thanks to its own optimising routines, can find the best value of the parameters {A, D, As, α} to fit observed concentrations with the minimum deviation and seems able to simulate reality for all the different morphologies.

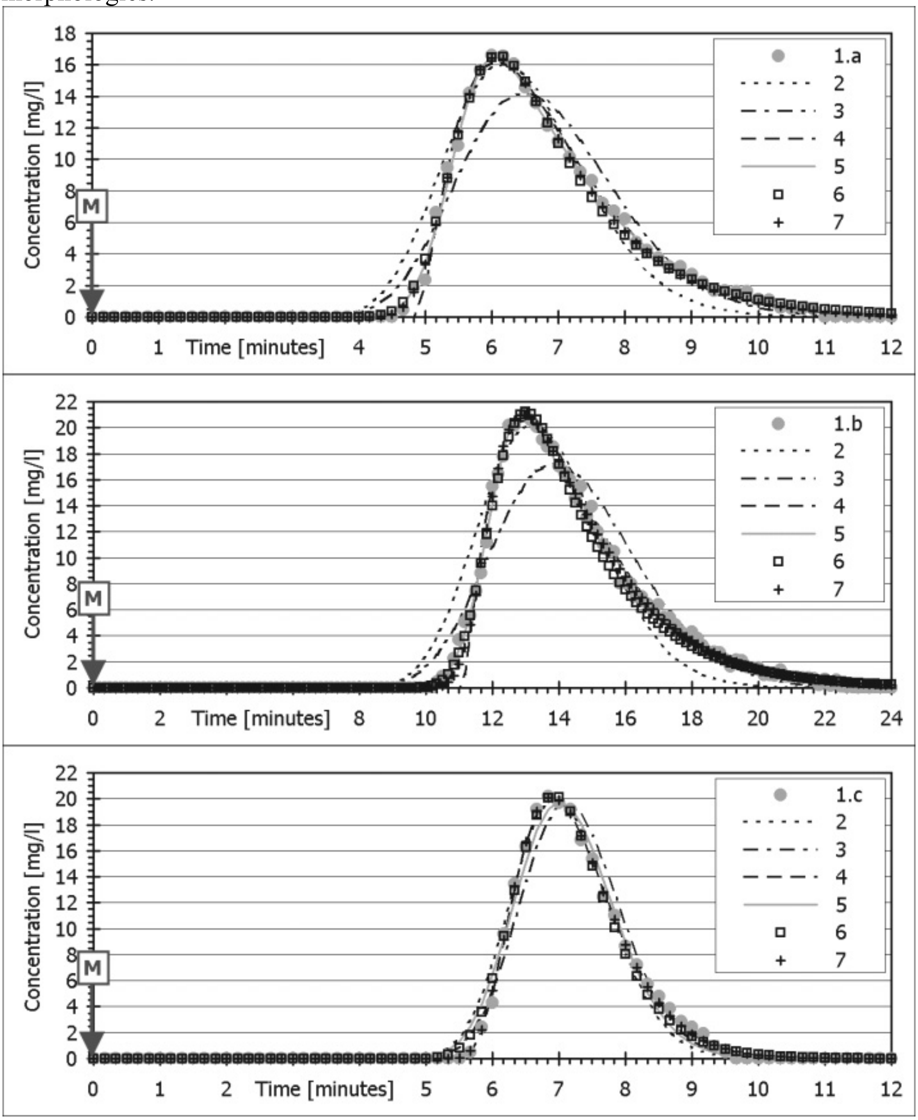


Fig. 6 - **1.a** Experimental data, in the highly meandering reach (2002/12/12), measured at cross

section 'II' after a spill of mass M = 3 kg at the input point 'S';

1.b Experimental data, in the winding reach (2002/10/24), measured at cross section 'G' after a spill of mass M = 3 kg at the input point 'D';

1.c Experimental data, in the straight reach (2002/07/17), measured at cross section 'M' after a spill of mass M = 4 kg at the input point 'N';

2 ADE solution optimised by the method of weighted least squares;

3 ADE solution optimised by the method of moments;

4 Shifted Gamma Distribution optimised by the method of weighted least squares;

5 OTIS (One-dimensional Transport with Inflow and Storage) model;

6 ADEK model (ADE solution combined with one on-line linear reservoir);

7 ADENK model (ADE solution combined with N on-line linear reservoirs).

The two proposed conceptual models ADEK and ADENK, despite their simplicity, are satisfactory enough in the simulation of the pollutant dynamics, showing a good fit of the field data collected in all the studied reaches. After the calibration of their parameters {A, D, K, (N)} by the method of weighted least squares, that is:

$$\min_{A,D,K,(N)} \sum_i [(C_{observed,i} - C_{ADE(N)K,i})^2 \cdot C^2_{observed,i}] \qquad (20)$$

they predict both the peaks and the persistence times of concentrations above a certain threshold value in a proper way. However, since the ADEK simulation seems to be good enough with just three parameters and since the curve simulated by ADENK does not show a great improvement in fitting the field data, it is not worth using the second one, which needs one more parameter.

5. Analysis of the parameters calibrated by field data as a function of discharge

After the calibration of all the models on each experimental wave, the optimised parameters have been analysed one at a time, and reach by reach, as a function of their respective constant stream flow rate (disregarding the Shifted Gamma Distribution, the parameters of which do not seem to have any immediate link with physical phenomena).

The average area A (Fig. 7) can be interpreted as the part of the cross section where the flow, and therefore the real advective-dispersive process, occurs. It is univocally related to the main channel velocity $A = Q/U$ and so it depends on the hydraulic characteristics of the reach, keeping a rising trend for an increasing discharge everywhere. Comparing the values of $U = Q/A$ derived from the calibration in the various models and the ones reported in Tab. 1 as directly calculated from the field data as $\overline{U} = x / ToT$, it seems that the models always make a certain overestimation, stronger in the meandering reaches, in comparison with the experimental values; probably because the models consider only the effective area and do not average the velocity with that in dead zones, where it is usually very low.

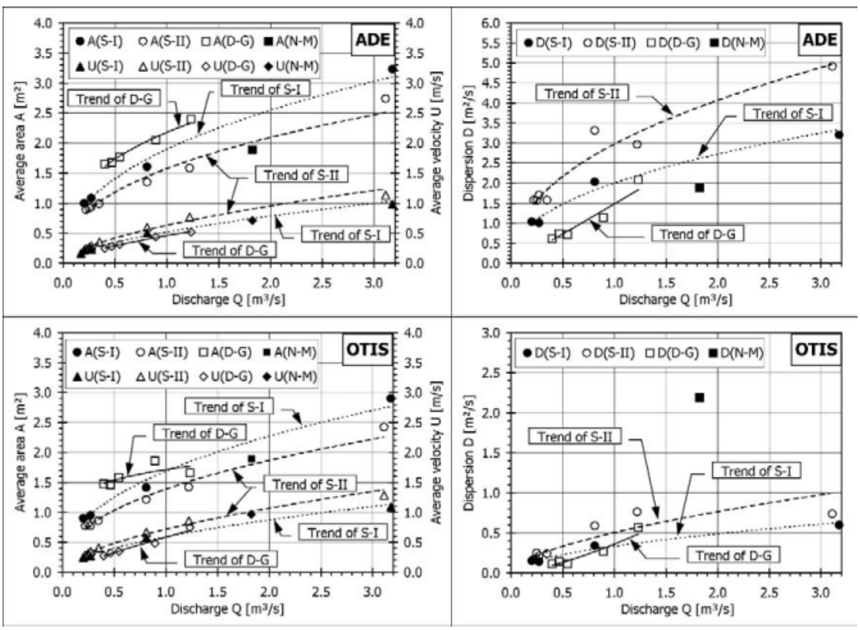
It is important to underline that the area A, which could seem a physically based quantity, actually must be considered as a model parameter.

Also the dispersion coefficient D (Fig. 7) is interpreted in a different way by the ADE solution with respect to all the other models. In fact, the analytical solution must take into account not only advection and dispersion but also the transient storage with just two parameters; thus D assumes values much higher than the respective ones in the OTIS or the ADEK / ADENK models, where specific parameters concern the third aspect. So, in the straight reach N-M, where the effect of the dead zones is negligible, it can be noticed that all the other models achieve their own best fit through D values comparable with the D value of the ADE solution.

The transient storage constant K (Fig. 8) has a decreasing pattern as a function of discharge in all reaches, due to the stronger influence of pools for low flow rates (with average deviation between K values in the various meandering reaches of about 10%).

Dead zones become less and less important with the increase of discharge, because of the turbulence that involves the pools and makes the mass exchange more and more effective. The same phenomenon can be seen through the analysis of the last two parameters of OTIS: the increasing influence of dead zones is not due to the variation of the ratio As/A, between ineffective and active flow areas in the cross section (As/A is about constant with discharge, even if at different values reach by reach), but depends on the increase of the storage zones exchange coefficient α.

The parameter N (Fig. 8) of the ADENK model is quite constant with discharge, but at different levels, reach by reach, being influenced by their different tortuosity.



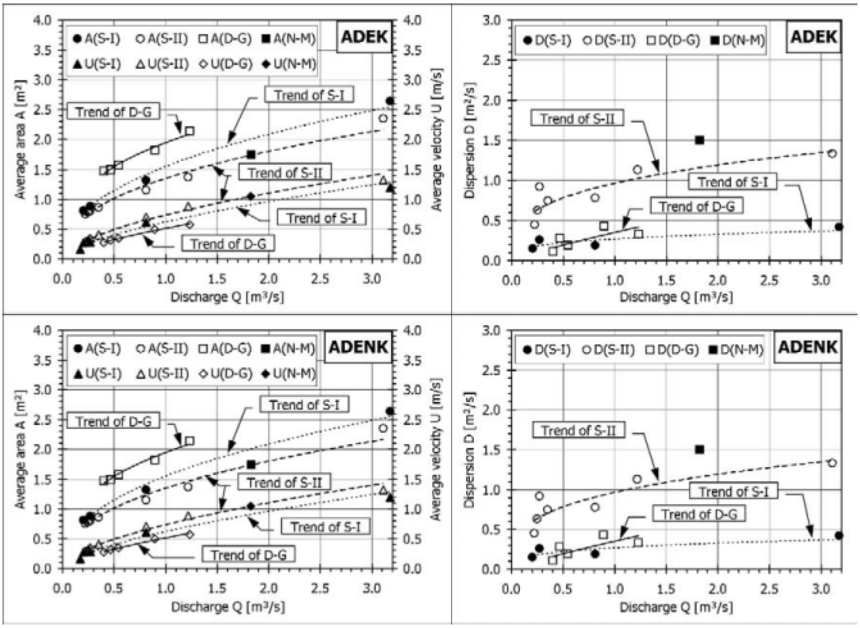
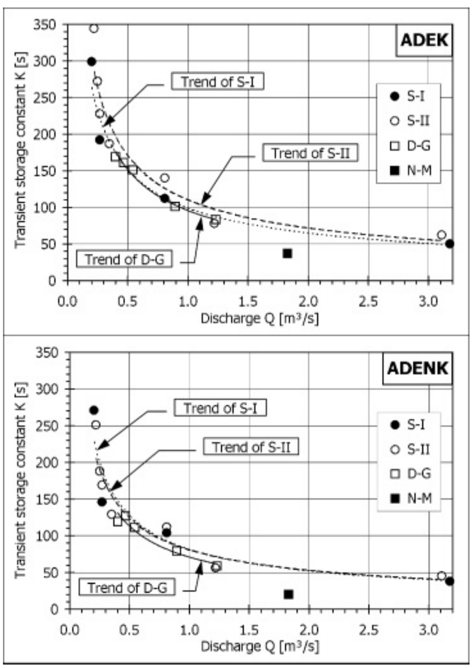


Fig. 7 - Average area A and velocity U vs. discharge Q, according to different models (on the left); dispersion coefficient D vs. discharge Q, according to the different models (on the right).



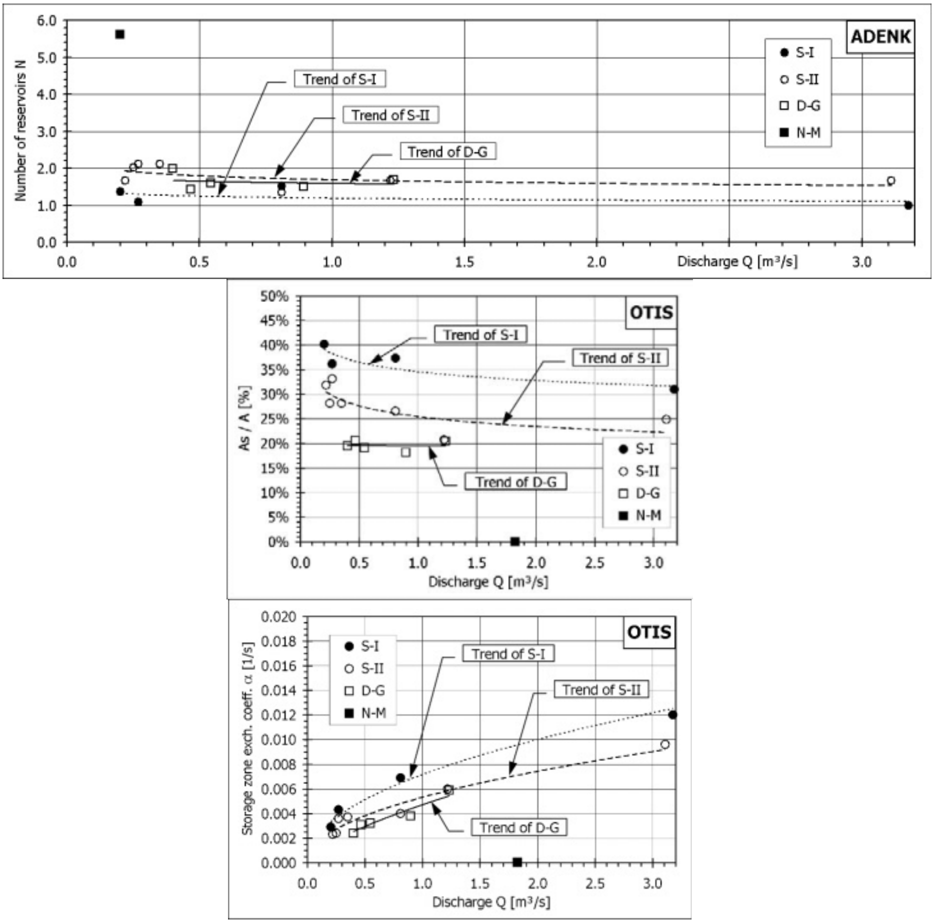


Fig. 8 - Transient storage constant K vs. discharge Q, according to both ADEK and ADENK (upper graph); Number of reservoirs N vs. discharge Q, according to ADENK (graph in the middle); Ratio between ineffective and active flow areas As/A vs. discharge Q, according to OTIS (lower on the left); Storage zones exchange coefficient α vs. discharge Q, again according to OTIS (lower on the right).

6. Validation of the ADESS approach (superimposition of effects)

The analysis has been extended also to the transport of pollutants caused by a non-impulsive input of pollutant, by means of the above mentioned ADESS model.

To verify the reliability of its simulation, three ToT measurements have been collected in the reach S-I-II: one in the reach I-II (concentration measured in the cross section 'II' after a first spill of a known mass M at the input point 'I'), one in the reach S-I (concentration measured in the cross section 'I' after a second spill of the same mass M at the input point 'S') and another contemporary one in the global reach S-II (concentration measured in the cross section 'II' after the same spill of a mass M at the input point 'S').

First of all, the three parameters of the ADEK model have been calibrated to get the best fit of the field data collected in reach I-II $\{A_{I\text{-}II}, D_{I\text{-}II}, K_{I\text{-}II}\}$ (Fig. 9).

Then the concentration at section ‘II’ has been simulated by the ADESS model, in terms of propagation downstream towards section ‘II’ of the experimental wave measured at section ‘I’ suitably discretised, using in the IUP_{ADEK} the same parameters derived from the previous analysis of the effects of a single spill in ‘I’ of the reach I-II (i.e., the values $\{A_{I\text{-}II}, D_{I\text{-}II}, K_{I\text{-}II}\}$ optimised through calibration).

Comparing this last simulation with the concentrations observed in the global reach S-II, the ADESS approach seems to be able to predict both the peaks and the persistence times of concentrations above a certain threshold value, with good agreement with the field data (Fig. 10).

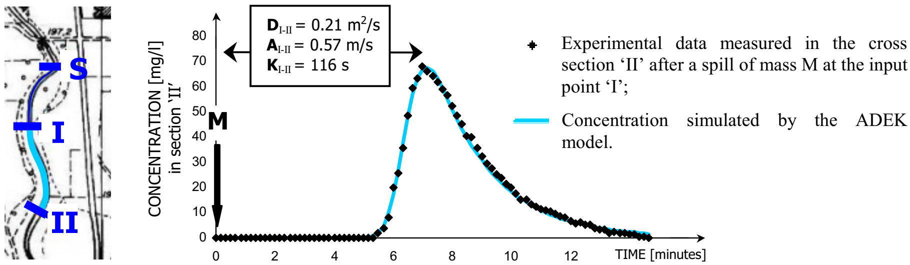


Fig. 9 - Field data (2003/04/01) interpreted with the ADEK model in the reach between cross section ‘I’ and cross section ‘II’ (the reach is shown in the map on the left).

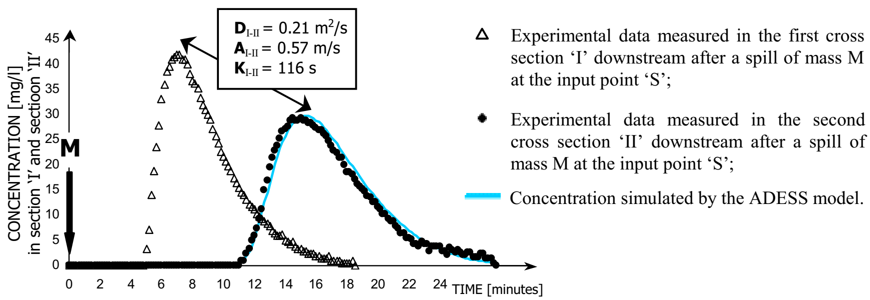


Fig. 10 - Field data (2003/04/01) interpreted with the ADESS model in the reach between cross section ‘I’ and cross section ‘II’.

7. Proposed conceptual models for non-conservative pollutants

The linearity of the solute decay, which is first-order, satisfies the requirements for applying both the superimposition of effects and the IUP approach, under the condition of modifying properly the conceptual ADEK and ADENK models described before.

The effect of the solute linear decay at time t, in case of an input with constant concentration, consists essentially in an exponential factor $e^{-\lambda \cdot t}$ multiplying the initial concentration C_0 to give:

$$C(t) = C_0 \cdot e^{-\lambda \cdot t} \tag{21}$$

Therefore the IUP_{ADEK} and the IUP_{ADENK} expressions must be modified as follows, now becoming called respectively $IUP_{ADEK\text{-}d}$ and the $IUP_{ADENK\text{-}d}$, in order to include the solute linear decay, both in their single reservoir or multiple reservoirs factor and in their ADE factor (Fig. 11):

$$IUP_{ADEK-d}(t) = \int_0^t \frac{Q}{2\cdot A\cdot\sqrt{\pi\cdot D\cdot\tau}}\cdot e^{-\frac{(x-U\cdot\tau)^2}{4\cdot D\cdot\tau}}\cdot e^{-\lambda\cdot\tau}\cdot\frac{1}{K}\cdot e^{-\frac{t-\tau}{K}}\cdot e^{-\lambda\cdot(t-\tau)}\cdot d\tau \tag{22}$$

$$IUP_{ADENK-d}(t) = \int_0^t \frac{Q}{2\cdot A\cdot\sqrt{\pi\cdot D\cdot\tau}}\cdot e^{-\frac{(x-U\cdot\tau)^2}{4\cdot D\cdot\tau}}\cdot e^{-\lambda\cdot\tau}\cdot\frac{1}{K\cdot\Gamma(N)}\cdot\left(\frac{t-\tau}{K}\right)^{N-1}\cdot e^{-\frac{t-\tau}{K}}\cdot e^{-\lambda\cdot(t-\tau)}\cdot d\tau \tag{23}$$

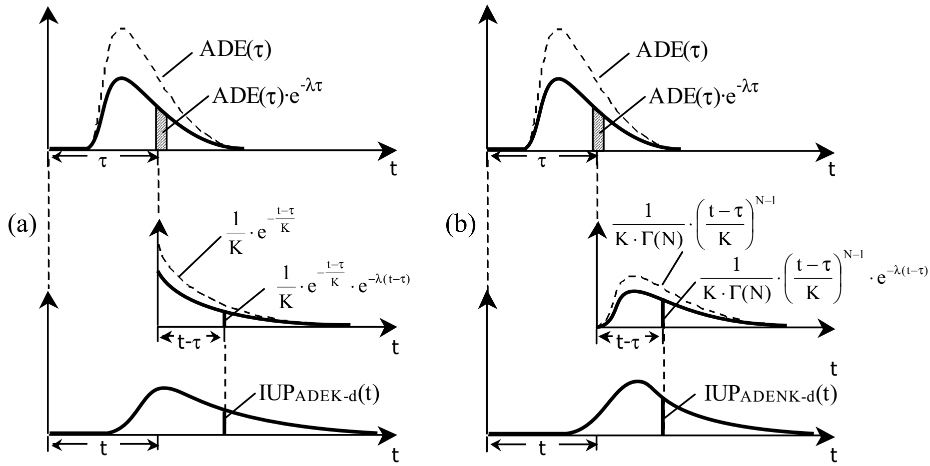


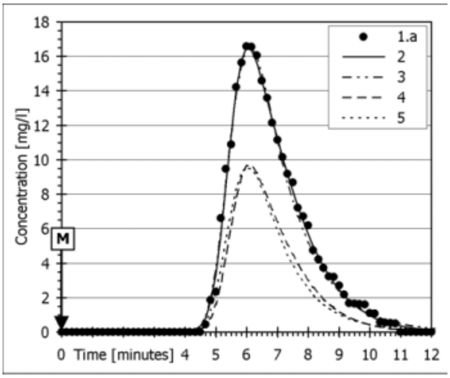
Fig. 11. Scheme of addition of the decay factor to: (a) the ADEK model, (b) the ADENK model.

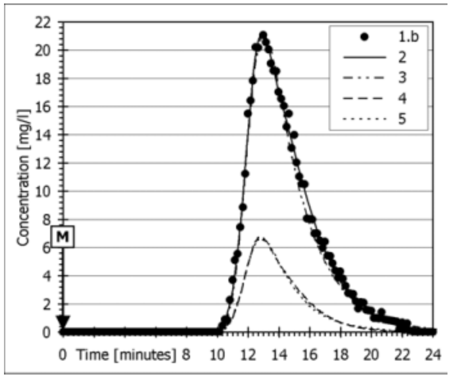
Of course, it is easy to see that these modified models can be used also for a conservative solute: in fact, when $\lambda = 0$ $IUP_{ADEK\text{-}d}$ coincides with IUP_{ADEK} and $IUP_{ADENK\text{-}d}$ coincides with IUP_{ADENK}.

8. Comparison of the ADEK-d model with OTIS for a non-conservative pollutant

To verify the reliability of the simulation obtained by the ADEK-d (and ADENK-d) model for a non-conservative pollutant, a comparison with OTIS has been made, keeping for each model the value of the parameters {A, D, K, (N)} and {A, D, As, α} estimated from the previous calibration on field data, and fixing the decay rate at the same value for both models, $\lambda = 0.0015\ s^{-1} \cong 130\ d^{-1}$. This value is much larger than the actual ones [26], but it has been chosen to emphasise the decay and consequently the differences between the two models (Fig. 12), since the distances and therefore the durations involved in the simulation are very short.

In all the different morphological situations the ADEK-d model fits the concentrations simulated by OTIS; so the model may be considered reliable also to describe the dynamics of non-conservative pollutants. Nevertheless it should be noted that for a limited duration discharge event the influence of the biochemical decay seems to be less important than the combined effect of dispersion and transient storage in dead zones.





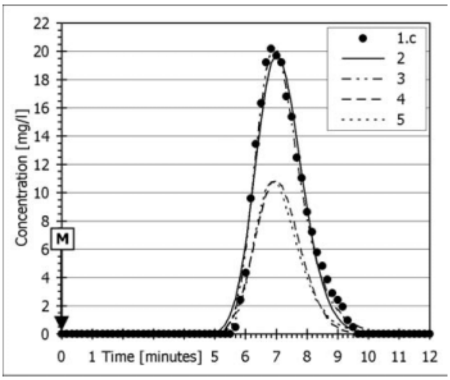


Fig. 12 - Comparison between OTIS and ADEK-d when simulating the same first order decay rate:

1.a Experimental data (2002/12/12) in the very meandering reach, measured at cross section 'II' after a spill of a known mass M = 3 kg at the input point 'S';

1.b Experimental data (2002/10/24) in the winding reach, measured at cross section 'G' after a spill of a known mass M = 3 kg at the input point 'D';

1.c Experimental data (2002/07/17) in the straight reach, measured at cross section 'M' after a spill of a known mass M = 4 kg at the input point 'N';

2 OTIS model calibrated for conservative solute;

3 ADEK model calibrated for conservative solute;

4 OTIS model with first order decay ($\lambda = 0.0015\ s^{-1}$);

5 ADEK-d model with first order decay ($\lambda = 0.0015\ s^{-1}$).

9. Conclusions

The models that simulate the propagation of pollutants in streams neglecting the influence of the transient storage in dead zones seem to be unable to fit the skewness of the data measured such reaches, like the meandering ones, where this phenomenon is important. Two conceptual models, the so-called ADEK and ADENK, combining the classic ADE solution for single instantaneous spill with the conceptual schemes of linear reservoirs well known in hydrology, have been proposed to simulate the one-dimensional propagation of rapidly transient pollutograph in streams with steady flow. Their effectiveness has been verified on the basis of 17 ToT surveys specifically collected in three reaches of the Lura stream, with different levels of importance of the dead zones. It has been verified also that they can simulate, by means of the so-called ADESS superimposition procedure, the pollutograph propagation through a series of reaches, each one assumed to possess homogeneous hydraulic and geometric characteristics. Moreover, the two proposed models have been compared with the classic ADE solution, the Shifted Gamma Distribution random walk method, and the OTIS model.

The ADEK and ADENK models can be physically justified, as some of their parameters show trends, which are related to the roles played by the phenomena of advection, dispersion and transient storage in dead zones, respectively.

They seem able to perform excellent simulations of the experimental data, working in a comparable or even better way than the physically based models. Also, they have been extended to the more general case of non-conservative solute with first order decay rate, becoming the so-called ADEK-d and ADENK-d. It has been verified that their results fit the OTIS outputs for the same decay rate.

Future developments of this research are expected from additional ToT surveys that at present are being collected, either in the Lura stream or in other ephemeral streams, like the Navile canal, which receives CSOs from the Bologna sewer system. The further work is also focused evaluating the feasibility of extending the ADESS procedure to unsteady flow.

10. References

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LOCAL SCOUR DOWNSTREAM OF GRADE CONTROL STRUCTURES IN URBAN STREAM RESTORATION

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1. Introduction: erosion in the urban reach of the Tevere River and its restoration

Urban streams need restoration structures, such as sills, in order to prevent excessive erosion of the bed, which could undermine the stability of other structures, such as bridge piers or embankments.

An interesting example of the restoration process performed on an urban stream is represented by that performed on the Tevere River in Rome. This process started in 1870, as soon as Rome became the capital of the Kingdom of Italy, and consisted in building high embankments along the urban reach of the Tevere, in order to contain floods. As a matter of fact, until 1870 no important works have been done on the Tevere, in order to prevent flood hazards, although important floods occurred 20 times from the 13th to 19th century (approximately three important floods every century) [1]. The restoration works lasted approximately sixty years; in 1930 the construction of the embankments along the urban stretch of the river, together with the implementation of other important works for the protection of the reclaimed lands in the proximity of the mouth of the river, was fully completed. As a consequence of the construction of the embankments and their quays, the erosion of the bed started to increase dramatically. Visentini [2] estimated that during the twelve years 1930-1942 the average stream bottom subsided 0.81 m. Later Torzilli et al. [3] showed that from 1942 to 1961, the erosion process continued without slowing down. The stability of ancient bridges was in danger and urgent measures had to be adopted to control the erosion process. In particular it was decided that several sills were to be positioned along the urban stretch of the river. As a consequence, ten years after building the first sill, in 1976, it was noted that the erosion process was not only arrested but even reversed [4]. The positive effects of the sills were however counterbalanced by local scour, which occurred downstream of the sills and which could affect the stability of both the structure itself and other structures in the vicinity (bridge piers, embankments, etc.). Some of the sills had to be removed and located in different cross sections of the river. It clearly appeared that the proper design and position of the sill, as is the case for other grade control structures as well, must account for the scouring process, which is determined by the flow over the sill structure and can seriously influence the stability of the latter.

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The determination of proper design criteria is the result of a research work, which can be performed both theoretically and experimentally. Incidentally, in this case, it is necessary to investigate the dynamics of the flow over the grade control structure and its interaction with the scouring process. However, such flow is very complex, non-uniform and turbulent. A complete and reliable physical-mathematical model of such a phenomenon is rather far away from being at the disposal of engineers. As a consequence, the investigations concerning local scour phenomena are usually conducted in laboratory models. Such a research trend is shown in many recent scientific works [5], [6], [7], in which experimental studies of local scour problems downstream of different structures (such as sills, energy dissipators, gates) are addressed, and it is followed in the present paper.

In particular, the aim of this study is to contribute to a better understanding of the scouring process downstream of a sill, followed by a rigid apron. Such measures are often applied in rivers, in order to reduce the local scour downstream of the control structure. The experiments were carried out in the Hydraulics Laboratory of the Department of Civil Engineering Sciences of the University RomaTRE. The temporal evolution of the scour profile, recorded by a CCD camera and obtained by image analysis techniques, was documented. The velocity field, measured by an ultrasonic velocity profiler at the end of the run, when the equilibrium state was reached, is also presented. Finally, an interpolation formula, obtained from the experimental data, is proposed.

2. Experimental set-up

The experiments were carried out in the hydraulics laboratory of RomaTRE University, using a 0.8 m tilting flume (17 m long, 1 m high and 0.8 m wide) with a rectangular cross section. The slope of the flume bed was close to zero. A test section, 0.3 m high and 3 m long, positioned 7 m downstream from the flume inlet, was created by artificially raising the flume bed. A uniformly graded sand, with a mean diameter d_{50} = 0.6 mm, was used to fill the test section, in order to create a mobile bed.

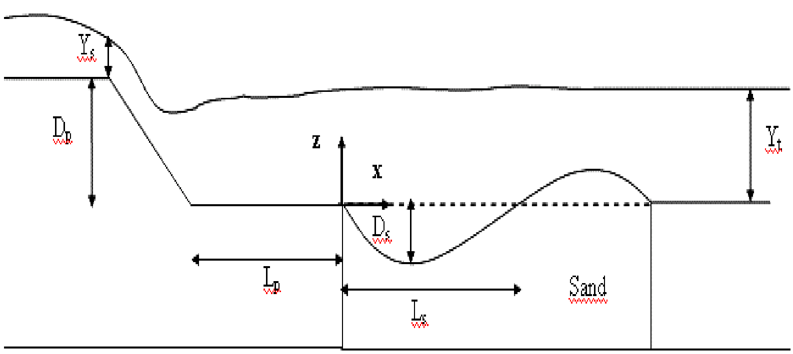

Figure 1. Definition sketch for model tests.

The same sand was glued both to the upstream and downstream fixed-bed sections, in order to form a bed with homogeneous roughness.

A sill, with a height of $D_p = 0.15$ m, followed by a $L_p = 0.5$ m long rigid apron, was positioned upstream of the test section (see Figure 1). A weir, positioned at the end of the flume, was used to vary the tailwater level.

Before starting the experiments, the level of the mobile sand bed, formed in the test section, was set flush with fixed bed, the weir was completely closed and the flume was slowly filled with water pumped from the laboratory supply reservoir. The discharge was slowly increased and set to the desired experimental value, carefully avoiding any movement of the sand bed. The experiment started at $t = 0$, after setting the tailwater depth rapidly to the desired experimental value by opening the weir. The discharge was maintained constant for the whole duration of the experiment.

The evolution of the scour hole was recorded on a mini DV cassette, using a CCD camera (Sony XC-77CE) connected to a digital videocassette recorder (Sony DSR-11). A detailed study of the temporal evolution of the scour hole was possible using image analysis techniques. The water level in flume was measured using a capacitance point gauge with a precision of 0.1 mm, mounted on a carriage moving along the flume.

At the end of the experiment, when the scouring process is assumed to have reached an equilibrium state, velocity measurements were performed by an ultrasonic velocity profiler [8]. The Acoustic Doppler Velocity Profiler (ADVP), first developed for medical studies of blood flow [9], is currently used to perform measurements both in transparent [10],[11],[12] and opaque fluids and high temperature fluids [13], giving spatial-temporal velocity information. The ADVP sensor is a piezoelectric probe, which works both as emitter and receiver of a signal consisting of ultrasonic pulses travelling across the fluid and back-scattered by the targets, i.e., small air bubbles or particles, moving with the flow. By evaluating the time lapsed between the emission of the pulse and its reception it is possible to determine the position of the target. The received backscattered pulses are shifted in frequency, due to the Doppler effect, so it is possible to calculate the velocity component of the flow along the probe axis by evaluating this frequency shift. For a detailed description of the ADVP technique see Takeda [14], [15].

In these experiments the pulses were repeated with a pulse repetition frequency (PRF) of 2,200 Hz and the number of emissions used to compute one profile is 40, corresponding to a final sampling frequency of about 25 Hz. The size of the measurement volume depends on the characteristics of the probe (emitting frequency and diameter) and increases away from the probe, due to the lateral spreading of the pressure wave. During these runs probes with an emitting frequency of 2 MHz and a diameter of 14 mm were used. The probe divergence angle was 1.83°. The ADVP allows a quasi-instantaneous measurement of the velocity in different volumes (gates), positioned along the axis of the probe. The distance between the centres of these gates used for the experiments was 3.52 mm. The longitudinal size used for the sampling volume was 2.96 mm, while its lateral size depended on the distance from the transducer. For the measurement of the profiles of the two velocity components, the horizontal and the vertical one, three probes with the same emitting frequency were used [16].

Velocity profiles were measured in different longitudinal sections (-25, -15, -5, 5, 15, 25, 35, 45, 55, 65, 75, 85, 95 and 105 cm following the reference system shown in Figure 1) in the centre of the flume. Due to the instrument configuration, the data can not be obtained in regions closer to the water surface or the eroded bed than 5 and 1.5 cm, respectively. For the same reason it was not possible to perform velocity measurements in a zone located approximately 25 cm upstream of the end of the rigid apron.

Measurements of the water surface elevation along the centre line of the flume were performed using a capacitance point gauge. At the end of the experiments, measurements of the bed profile were made along the longitudinal axis of the flume.

3. Results

3.1 GENERAL CONSIDERATIONS

Several experiments were performed, each characterised by a different discharge. In total eight discharge values were considered, as summarised in Table 1.

Table 1. Summary of the experimental runs

Run	Q [l/s]	Y_s [m]	Y_t [m]	D_s [m]
1	11.0	0.021	0.126	0.042
2	12.3	0.024	0.129	0.044
3	14.2	0.023	0.134	0.050
4	16.2	0.023	0.137	0.061
5	18.4	0.028	0.143	0.067
6	20.5	0.029	0.145	0.069
7	24.8	0.032	0.151	0.077
8	25.8	0.034	0.156	0.082

The meaning of the symbols is the following: Q is the discharge, Y_s is the tailwater depth over the sill, Y_t is the tailwater depth downstream of the scour, and D_s is the maximum scour depth.

All experiments were performed in clear water. After the desired tailwater depth was reached, the formation of a submerged hydraulic jump was observed on the rigid apron immediately downstream of the sill. This is obviously a desired configuration, in order to reduce the local scour. The scouring process started contemporaneously with the formation of such a hydraulic jump and was characterised by a very rapid increase during early part of the experiment, followed by a slower and slower increases, until the equilibrium state was reached, defined as that condition in which no sediments moved. From a phenomenological point of view the scouring process is caused by a jet, which is caused by the presence of the sill. In fact, the flow accelerates on the sill, where the Froude number is always greater than 1, and forms a submerged hydraulic jump over the rigid apron. The presence of a wall jet over the apron was noted. Such a wall jet contacts the mobile bed and starts the scouring process, which, due to the high velocity, progresses very rapidly. However, while the scouring process progresses, the wall-jet becomes a free-jet, whose maximum velocity does not correspond to the mobile bed [17]. For this reason, the scouring process slows down, until equilibrium state is reached, in which no sediments move. Such a phenomenological description holds true for all valid experiments.

Detailed analysis of the velocity field as well as of the temporal evolution of the scouring process was performed for each valid experiment. In the following, analysis of run No. 4, characterised by a discharge of 16.2 *l/s*, will be presented.

3.2 THE VELOCITY FIELD

The velocity field measured by the ADVP, the bed profile measured in the centre of the flume and the free water surface profile are shown in Figure 2. The horizontal and vertical distances are made dimensionless by dividing them by the water depth over the sill, Y_s.

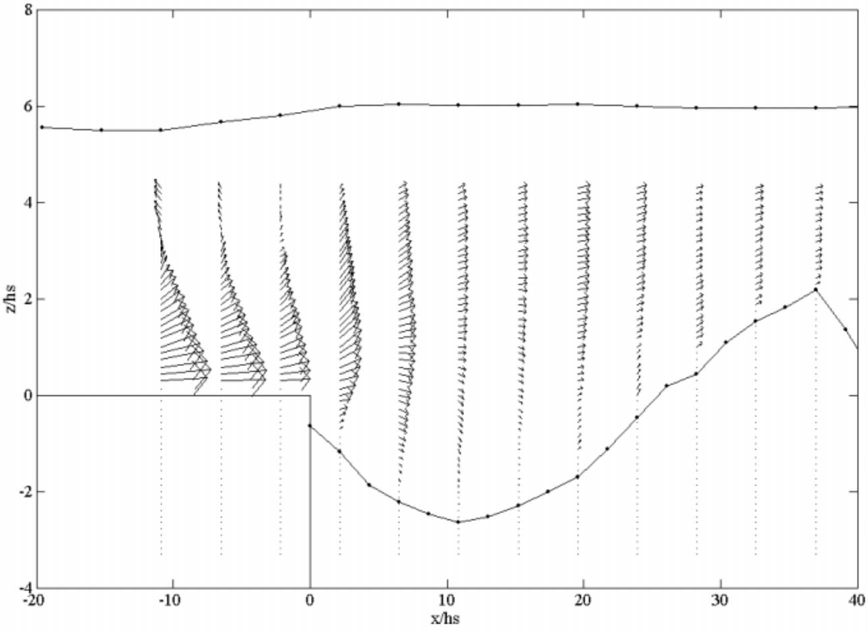

Figure 2. The velocity field.

The jet created by the flowing water over the sill first moves over the rigid apron, positioned immediately downstream of the sill, with a wall-jet velocity distribution. It is possible to observe the presence of a reverse flow close to the water surface over the rigid apron. This reverse flow is due to the presence of the submerged hydraulic jump, positioned over the apron. The reverse flow becomes stronger and stronger, decreasing with the distance from the sill. The flow which moves over the mobile bed has a free-jet velocity distribution and its maximum velocity continues to decrease with increasing distance from the sill. In a zone between the downstream end of the rigid apron and about $L_{max}/2$, where L_{max} is the length of the scour, the velocity vectors close to the mobile bed are small. Moving downstream of the maximum depth, the velocity vectors start to attain a constant positive value (considering the frame of reference shown in Figure 1) over the whole depth. The flow in this region is oriented slightly upward, due to the upward slope of the eroded-deposited bed.

3.3 TEMPORAL EVOLUTION OF THE SCOUR HOLE

The temporal evolution of the scour hole, from the beginning of the experiment, was recorded on a mini-DV, using a CCD camera and a videocassette recorder. Applying an image analysis technique, it was possible to measure the temporal evolution of the scouring process at the wall of the flume. This technique has the advantage of making scouring measurements in many different points simultaneously, which is important for studying the fast evolution of the scour during the first minutes from the beginning of the run. The disadvantage of the technique is that the scour on the glass wall is always bigger than the scour in a plane in the centre of the flume. However, different measurements of the bed profile, made in 5 different transversal planes of the flume, showed that the bed profile was approximately 2D, with the differences between measurements close to the wall and others in the centre of the flume being very small compared to the total eroded depth. Thus, it was possible to use the data obtained by the camera as a reasonable estimate of the scouring process.

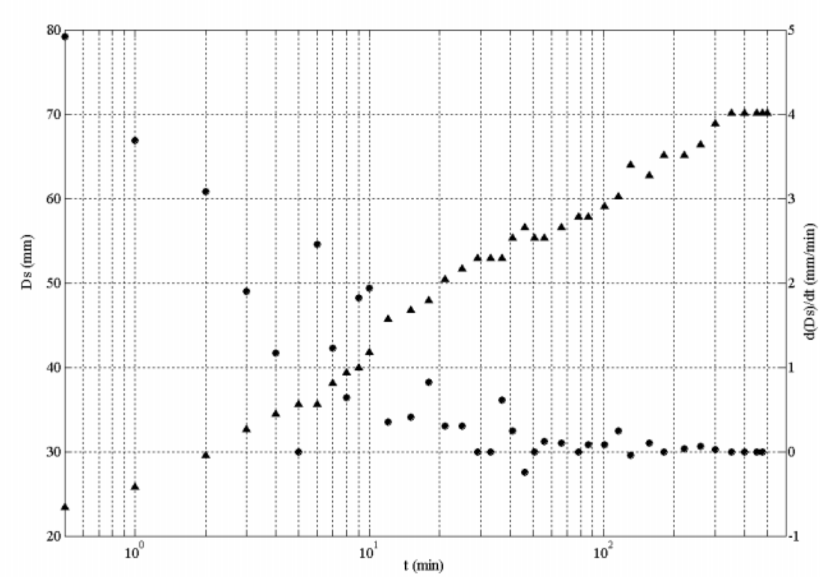

Figure 3. Temporal evolution of the maximum scour depth (triangles) and time rate of its growth (dots).

In Figure 3 the temporal evolution of the maximum depth of the scour (triangles) and the time rate of the scour (circles), representing the measurements performed during the run, are plotted. It is possible to observe how the scouring process evolves very fast during the first minutes of the experiment and then it progressively slows down in time. The equilibrium state has been reached after 351 minutes (5.85 h) since the start of the experiment. After that time, no sediment transport was observed throughout the measuring section. The maximum scour depth measured was 0.061 m and the length of the scour hole was 0.60 m. Almost 90% of the total scour depth occurred during the first 28% of the total time duration. A similar behaviour was observed by others [7].

In Figure 4 the temporal evolution of the scour profiles, measured by the camera, is plotted. Six different profiles, measured at 1, 4, 16, 66, 260 and 501 minutes, are plotted. After only 1 minute a scour hole with a maximum depth of 0.025 m is formed and it possible to observe the presence of a small dune downstream of the hole. A lot of suspended material was observed during the first minutes of the run. It is possible to distinguish two different phases of the scour evolution. During the first phase, which ends at a time of about 351 minutes, when the scour hole reaches the equilibrium state, the scour profile changes very fast, increasing both in depth and length. The dune increases its height and length, moving downstream. During the second phase the maximum scour depth is always the same, while the scour hole continues its developments by increasing its length. During the evolution of the scour hole, it is possible to observe that the point of maximum scour depth was moving downstream.

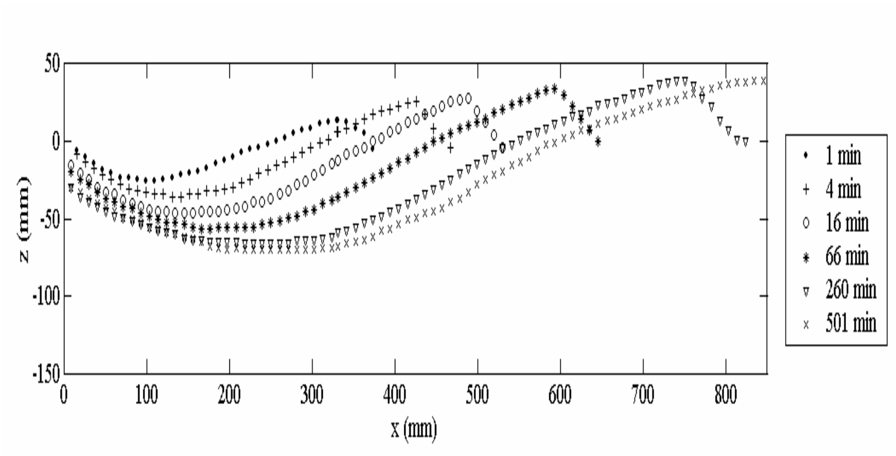

Figure 4. Temporal evolution of the scour profile.

3.4 INTERPOLATION FORMULA

In order to have a general idea of the behaviour of the studied phenomenon, a formula, which interpolates the experimental results, is proposed. Such a formula is obtained by applying the dimensional analysis to the studied problem yielding a finding that the maximum scour depth at the equilibrium state can be expressed as a function of proper dimensionless parameters. In particular, following the hypothesis of Mossa [18], the dimensionless maximum scour depth at the equilibrium state $\frac{D_s}{Y_t}$ is expressed as:

$$\frac{D_s}{Y_t} = f\left(Fr_d, Fr_s, \Delta H\right) \tag{1}$$

In eq. (1) $\frac{D_s}{Y_t}$ is the ratio of the maximum scour to the tailwater depth, Fr_d is the densimetric Froude number, Fr_s is the Froude number at the sill, and ΔH is the ratio of the difference between the tailwater depth related to the sill and the tailwater depth downstream ($\Delta H = \frac{Y_s + D_p - Y_t}{Y_t}$). The latter quantity gives the variation of the hydraulic grade line, due to the flow crossing along the sill. The densimetric Froude number is defined as:

$$Fr_d = \frac{Q}{bY_s\sqrt{\psi\left(\frac{\rho_s - \rho}{\rho}\right)gd_{50}}} \tag{2}$$

where $\psi = f\left(\frac{u_* d_{50}}{\nu}\right)$, the Shields parameter, is a function of $Re_* = \frac{u_* d_{50}}{\nu}$. In these relations d_{50} is the characteristic diameter of the sediment, and ρ, ρ_s are, respectively, the density of the water and the sediment. In the present work u_*, the shear velocity, is calculated as:

$$u_* = \frac{nQ\sqrt{g}}{bY_s^{7/6}} \tag{3}$$

n being the Manning factor, which according to Krishnappan [19] is closely related to d_{50} by means of the following formula:

$$n = 0.13\frac{\left(2d_{50}\right)^{1/6}}{\sqrt{g}} \tag{4}$$

As a matter of fact, the values assumed by Re_* for each experiment are in the range: $3.93 \le Re_* \le 7.18$, for which the Shields parameter is approximately constant ($\psi \approx 0.035$).

The Froude number Fr_s is defined by adopting the water depth, measured in the first section of the sill, as the characteristic length:

$$Fr_s = \frac{Q}{b h_s \sqrt{g h_s}} \tag{5}$$

By adopting a monomial formula:

$$\frac{D_s}{Y_t} = K\, Fr_d^{\ \alpha} Fr_s^{\ \beta} \Delta H^{\gamma} \tag{6}$$

it is straightforward to apply a least-squares approximation to determine the coefficients K, α, β, γ.

The following values were obtained: $K \approx 2.9\,10^{-5}$, $\alpha \approx 3$, $\beta \approx -1.6$, $\gamma \approx 0.57$, which highlights the most important features of the studied phenomenon: the maximum scour depth increases with the discharge and decreases with the tailwater depth downstream.

The validity of this interpolation is shown in Figure 5, where the interpolated values of D_s / Y_t are plotted versus the experimental ones.

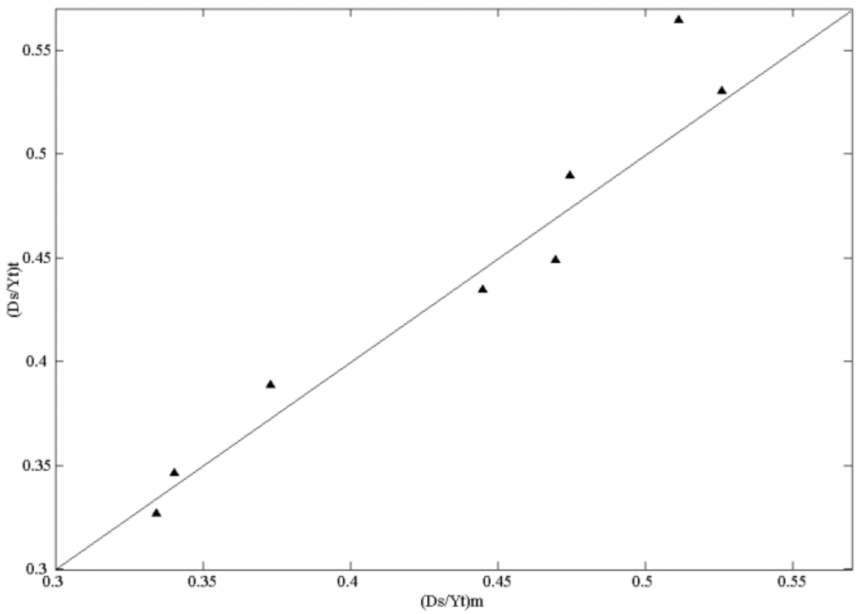

Figure 5. Comparison between the experimental and predicted values of the dimensionless maximum scour depth.

4. Conclusions

The local scour downstream of a grade control structure with a rigid apron was experimentally investigated in the hydraulics laboratory of the University RomaTRE. The evolution of the scouring process was recorded by a CCD camera and the profiles of the scour hole were obtained using image analysis techniques. The experiments show that the temporal evolution of the maximum scour depth is very rapid during the first minutes of the run and then decreases with increasing time. Velocity profiles, both the horizontal and the vertical components, were measured in the centre of the flume when the equilibrium scour depth was reached.

The velocity measurements show the presence of a wall jet, which moves downstream along the rigid apron, and is related to the submerged hydraulic jump. Such a wall jet is responsible for the initial rapid growth of the scour. As soon as the scour increases, a transformation of the wall jet into a free jet occurs and the scouring process tends to attain an equilibrium state, because the scouring effectiveness diminishes.

A formula obtained by interpolating the measured maximum scour depths is proposed. Such a formula indicates that the maximum scour increases with the discharge and decreases with the downstream tailwater depth.

As a concluding remark it can be stated that the present work has showed the main phenomenological characteristics of the local scour process. In particular, it highlighted that the presence of the rigid apron influences the maximum depth of the scour as it reduces the magnitude of the velocity corresponding to the mobile bed. The length of the apron is then one of the most important parameters for designing the structure, when the need to reduce the scour downstream highly important.

Further development of this work could consist in the development of a numerical model for simulating the temporal evolution of the scour hole or investigations of the rigid apron's length as an experimental parameter, in order to derive quantitative relations between the apron length and the maximum scour depth.

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UPGRADE OF WATER AND SEWERAGE FACILITIES IN SLOVAKIA TO ACHIEVE COMPLIANCE WITH MAJOR EU DIRECTIVES

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1. Introduction

Urban water services in Slovakia (territory of 49,014 km^2 and population of 5.4 million) include water treatment and supply, collection and treatment of wastewater and protection of receiving waters. Such objectives are examined, focusing on the challenges introduced by the need for compliance with major European directives, such as the Water Framework Directive 2000/60/EC, the Drinking Water Quality Directive 98/83/EEC and the Urban Waste Water Directive 91/271/EEC. The paper presents an overview of organisation of water resources management in Slovakia as well as examples of integrated solutions to drinking and wastewater problems in different regions of the country, as developed during preparation of project proposals under EU grant schemes, such as ISPA or the Cohesion Fund.

2. Water Resources

2.1 GROUNDWATER

Groundwater on the territory of Slovakia is considered extremely important because it represents the major source of drinking water. Although the hydrogeological conditions of accumulation, circulation and production of groundwater are generally favourable, the uneven distribution of groundwater resources and the fact that there are some hydrogeological structures practically devoid of useable groundwater supplies represent a disadvantage. Most of the documented useable groundwater resources are in the Western Slovakia region (56% of the total), found in Quaternary sediments of the Danubian Lowland and the alluvia of the Vah River and its tributaries. On the other hand, some of the Northern (Carpathian flysh-belt) and Eastern Slovakia regions have significantly lower documented volumes of useable groundwater (17% of the total). The remaining 27% represent the resources of the Central Slovakia region.

2.2 SURFACE WATER

The average amount of precipitation in the country is around 760 mm, with the Danubian Lowland being the driest (below 550 mm) and the High Tatras the most humid part of the country (> 2,000 mm). In dry periods, the capacity of surface water sources amounts to about 90 m^3/s. When subtracting ecological discharges (minimum discharge needed for supporting and maintaining aquatic life and other functions of a stream), only 37 m^3/s are available for utilisation (excluding the Danube, Morava and

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Tisza rivers). Water reservoirs across Slovakia allow increasing the discharges in dry periods by 54 m^3/s, thus increasing useable discharges to 90 m^3/s.

At present, there are 54 water reservoirs across Slovakia (with the overall capacity exceeding 1 million m^3 each) with the gross controllable capacity of 1,890 million m^3. The capacity of these reservoirs allows for the interception of about 14% of the annual mean discharge as well as for the increase of low discharges during dry periods by about 56 m^3/s (above the discharge Q_{355} = 80 m^3/s) [1]. The above mentioned water reservoirs include 8 reservoirs built for drinking water supply (see Figure 1). Their major purpose is to ensure large-scale drinking water supply of Northern, Central and Eastern parts of the country. During the most unfavourable dry periods, i.e., during the decreased yield of groundwater sources, these reservoirs (after treatment) can provide approximately 2,300 l/s of good quality drinking water, ensuring the highest level of water supplies, for about 270,000 inhabitants.

3. Quality of Surface Waters and Groundwater

3.1 SURFACE WATER QUALITY

The quality of surface water in the Slovak Republic is assessed by the use of major indicator groups starting with physical indicators, followed by chemical and radiological indicators, and ending with biological and microbiological indicators. Depending on the quality of water, streams are classified into five categories. It is notable that since 1990, the percentage of the worst quality (category V) has rapidly decreased at the sampled sites. While in 1990, 19% (618 km) of the sampled sites were classified as category V, in 1995 this has fallen down to 5 and 6% (160 km). This level has not changed significantly since then.

Besides this positive development with respect to physical and chemical pollution, as a result of unsatisfactory wastewater treatment and discharge of untreated wastewater into the rivers, biological and microbiological pollution of surface waters in Slovakia is still significant. Categories V and IV are repeatedly observed in the majority of monitored sites with over 80% of all sites falling into these categories.

3.2 GROUNDWATER QUALITY

Groundwater quality is monitored in over 25 areas that are considered significant in terms of water management, mainly in river alluvia and in Mesozoic and neo-volcanic complexes. The ongoing monitoring has proved that in monitored areas problems are emerging with respect to negative oxidation-reduction conditions in groundwater. This has been proven by frequently increased concentrations of Fe, Mn and NH_4. Compared to previous years, the pollution from organic substances (indicated by frequent exceedances of marginal concentration limits of non-polar extractable substances, COD and phenols) has remained the same. The prevailing type of land use (urban or/and agricultural areas) is reflected in the relatively frequent increases in the parameters of oxidised and reduced forms of nitrogen in the water.

4. Water Resources Protection

Water resources management/protection in Slovakia is in conformance with the requirements of the Water Framework Directive 2000/60/EC (WFD). Water resources protection is viewed as an integrated protection of quality and quantity of surface water and groundwater. The major determinant in terms of water resource *quality* protection is the water pollution source, having either direct or indirect impact on water resources. Water *quantity* protection is based on increasing the cumulative capacity of watersheds and aquifers and on controlling the observance of calculated values for the volumes of withdrawn water. This has been done by establishing groundwater utilisation limits (ecological limits), as well as binding minimum discharges (ecological discharges) in streams.

Both aspects of water protection (quantitative and qualitative) are subordinated to a *territorial water protection* system, particularly in such source areas that are considered significant from the perspective of water management. The system consists of three types of protection: *general protection*, resulting from the new Water Act No. 184/2002; broader *regional protection*, implemented by means of protected water management areas; and *special protection* with increased severity – a specific protection of drinking water sources implemented by means of hygienic protection zones [2].

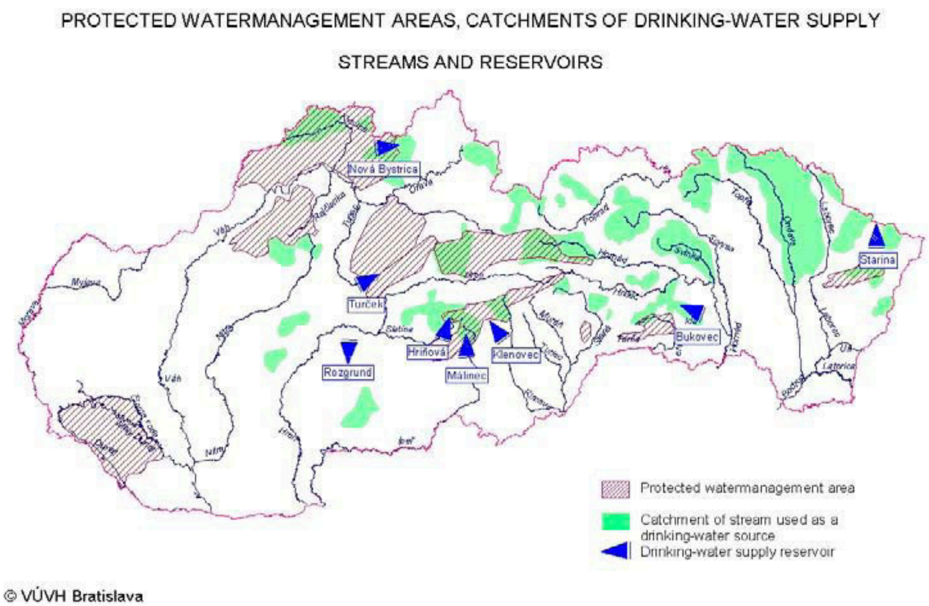

Figure 1 Protected areas of drinking water resources in Slovakia

The administrative bodies responsible for management of both surface and groundwater resources are the River Basin Management Authorities (RBMA) headed by the State Water Management Enterprise. These authorities, together with the

Slovak Hydrometeorological Institute (SHMI), which carries out monitoring, are responsible for administration of the five large river basins (hydrological units) on the territory of the country (Danube, Váh, Hron, Bodrog and Hornád) and thus provide complex operation and maintenance, protection against pollution as well as other services. Water quality is furthermore monitored by the State Inspectorate of the Environment.

5. Public Water Supply

5.1 CONNECTION RATIO AND WATER CONSUMPTION

The total number of inhabitants connected to public water supply network in 2002 was about 4.5 million inhabitants (84%). The highest share of those supplied by water from public water systems is in the Bratislava region; the regions of Trenčín, Žilina and Banská Bystrica also exceed the Slovak average. The population supplied by public water systems in the Košice and Prešov regions is below the Slovak average. The drinking water supply situation is even more disproportionate when broken down by districts, since the share of population supplied with drinking water varies between 50% (Vranov nad Topľou, Sabinov, Bytča, Košice) and 100% (Bratislava, Prievidza, Martin, Banská Bystrica, Partizánske and other).

Since 1991, water consumption in Slovakia has been constantly decreasing. This phenomenon is caused partially by decreasing industrial production, but also by the gradual decrease in public water consumption due to the increasing drinking water tariffs. The specific demand for drinking water has decreased since 1990 by almost 40 %; the current consumption value is 115 l/cap/day. Compared to EU countries, which consume approximately 145 l/cap/day, the specific water demand in Slovakia is low. The consumption of drinking water is falling deep below the average, leaving us dangerously close to approaching the minimal hygienic standards. This trend also applies to other water use categories, since the demand of industry and other users – compared to 1990 – has decreased by 30%. The amount of unaccounted-for water (UFW) is at the level of 29% of the produced water; out of this amount more than 97% covers losses in the distribution network.

5.2 REGIONAL (LARGE-SCALE) WATER SUPPLY SYSTEMS

Non-uniform occurrence of water in time and space in various regions of the country has led in the past to the development of large-scale water supply systems (see figure 2). Due to this, currently there are 47 regional water supply networks supplying some 70% of the inhabitants with good quality drinking water (i.e. 80% of all produced water in the country). Interconnections of the regional systems have resulted in the formation of supra-regional systems, such as the Western Slovakian, Central Slovakian or Eastern Slovakian Water Supply Systems, based on combined use of several large groundwater resources, and other systems are emerging (e.g. the Northern Slovakian and the Tatra Water Supply System). It is anticipated that future development in this area will focus on utilisation of existing and developed capacities over exploitation of new resources. The most suitable resources with enormous capacity are the ones located under the Žitný ostrov region. On the contrary, there are almost no resources in the Eastern part of the country.

5.3 DRINKING WATER QUALITY

Quality of drinking water in public water supply systems is assessed on the basis of inspections made by the operators of such systems (i.e., the water companies). We can state that over the last decade, despite some changes in the limits of some indicators, the total share of analyses meeting the STN limits remained at 98%. Among inorganic indicators, antimony and arsenic proved to be the most problematic ones. Before July 1998, antimony was not subject to monitoring and the permissible limit of arsenic was decreased from 50 µg to 10 µg/1 litre. At present, nearly 110 water treatment plants operate in the Slovak Republic producing drinking water by treating both surface water and groundwater. These water treatment plants treat approximately 15-16% of the total volume of water (4,000 – 4,500 l/s).

5.4 FUTURE DEVELOPMENT

The priority in the field of water supply for the future is to increase the proportion of population connected to public water supply systems to the level of the EU countries and to achieve a 97% connection ratio. The overall budget needed to secure this goal is estimated at SKK 70 billion (EUR 1.7 billion). It is clear that this can only be achieved with foreign assistance, i.e., through the pre-accession instruments or via the Cohesion Fund and structural funds.

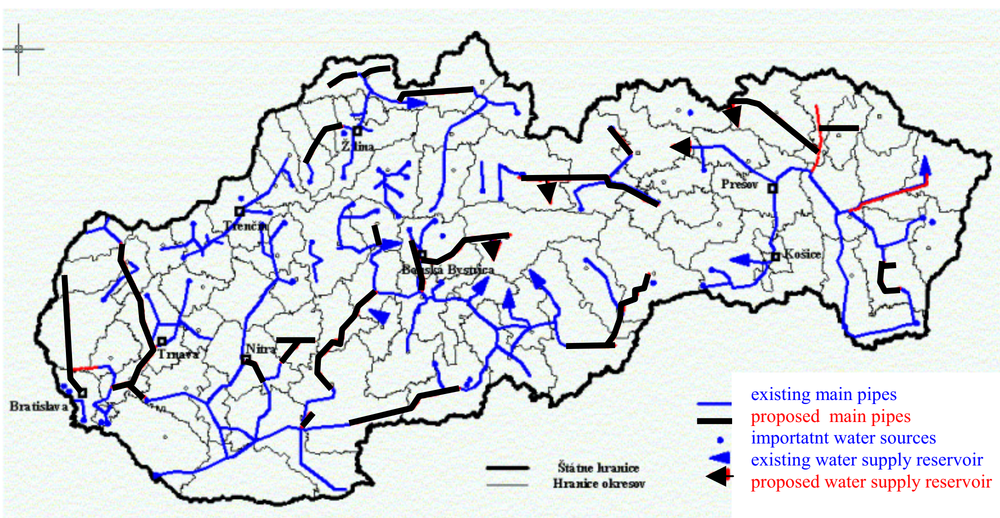

Figure 2. Regional water supply systems in Slovakia

6. Wastewater Collection and Treatment

6.1 GENERAL OVERVIEW

Out of the 2,883 municipalities in Slovakia only 490 are connected to public sewerage systems, i.e., 17%. Development of public sewerage systems lags behind the development of public water system networks (see Figure 3). Out of total volume of water produced and delivered to the customers through public water supply network, only 71% is later drained by public sewerage and only 67% is treated in existing wastewater treatment plants (WWTPs).

Nevertheless, and mainly due to the Phare and ISPA programmes, there is an evidence of a considerable number of sewerage systems and WWTPs under reconstruction and further systems being constructed.

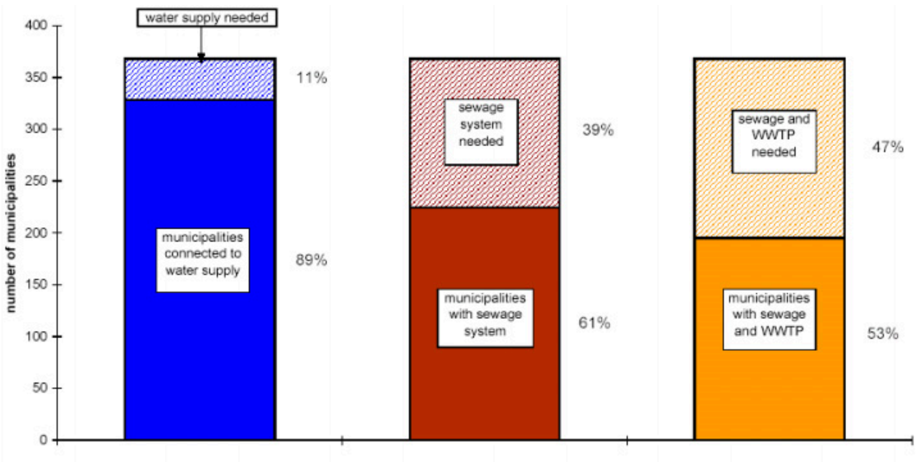

Figure 3. Difference between water supply and sewerage in Slovakia [1]

The Common Position of the European Union in Chapter 22 on the Environment, approved by the EC and SR at the end of 2001, appreciates in the part on Water Quality the fact that the Slovak Republic will designate the entire territory as “sensitive” in the sense of Article 5 of the Directive 91/271/EEC. The EU notes that advanced treatment will be provided for each treatment plant above a population equivalent of 10,000, i.e., that nitrogen removal will be ensured in accordance with Annex I of the Directive and that phosphorus removal will be ensured where specific circumstances of waters so require. This agreement is implemented in the Government Regulation 491/2002 (see bellow).

6.2 GOVERNMENT REGULATION No. 491/2002

General surface water quality objectives and effluent standards in Slovakia are set in the Government Regulation No. 491/2002 and applied by the local Environmental Agencies when issuing discharge permits.

The Regulation was prepared with the aim to agree with the European legislation, especially with the Council Directive 91/271/EEC. It represents a fusion of ambient water quality standards and end-of-pipe effluent standards common in European countries:

- *effluent standards* are defined both for municipal WWTPs and selected industrial wastewaters (in accordance with the UWWT Directive),
- *ambient water quality standards* are defined for the receiving water (see Table 1).

Water authorities have the right to set stricter effluent standards calculated from the "mixing equation" based on the ambient standards.

Table 1. Selected ambient standards in GR 491/2002, Annex 1 & Annex 2 (in mg/l)

Parame -ter	**General req.**	**Surface water utilised for drinking water supply**					
		Category A		**Category B**		**Category C**	
	RV	**RV**	**LV**	**RV**	**LV**	**RV**	**LV**
BOD_5	7	< 3.0	3.0	4.0	5.0	5.0	7.0
COD_{Cr}	35	10	15	15	25	25	35
SS		25					
$N\text{-}NH_4$	1.0	0.04	0.4	0.4	0.8	0.8	2.3[1)]
N_{tot} [2)]	8.5	1.5	11.4[1)]	8.9	11.8[1)]	10.5	14.2[1)]
P_{tot}	0.4	0.1	0.1		0.2	0.4	

1) Concentration values must not be reached under certain geographical conditions
2) Value calculated as sum of organic and inorganic nitrogen ($N\text{-}NH_4$+$N\text{-}NO_2$+$N\text{-}NO_3$)
RV – recommended limit value
LV – maximum limit value
Category A - water requiring physical treatment and disinfection or rapid sand filtration and disinfection
Category B - water requiring physical/chemical treatment and disinfection (e.g., coagulation, flocculation, filtration, disinfection by chlorine)
Category C - water requiring intensive physical/chemical treatment and disinfection (e.g., coagulation, flocculation, filtration, adsorption, disinfection by chlorine or ozone)

6.3 FUTURE DEVELOPMENT

The transitional measures that have to be completed after the accession of Slovakia to the EU, stated in Chapter 22 of the Common Position of the EU, are presented in Tab. 2.

Table 2. Transitional measures required under Directive 91/271/EEC

Agglomeration (AGM)	**Number of AGMs**	**Proposed implementation**	**Trans. period** (from 1/1/2004)
Sewerage systems			
AGMs with >10,000 PE	18	2010	6 years
AGMs with 2,000 –10,000 PE	409	2015	11 years
Wastewater treatment			
AGMs with >10,000 PE	90	2010	6 years
AGMs with 2,000 – 10,000 PE	439	2015	11 years

7. The ISPA Programme and the Cohesion Fund

To facilitate the efforts of candidate countries in the field of wastewater treatment, the European Union has strengthened its support to the accession countries through the ISPA instrument, based on the Council Regulation No. 1267/1999. In the Accession Partnership 1999, the Slovak Republic proposed the framework of priority areas for further preparatory work in the accession process towards membership in the European Union. The National Programme for the Adoption of the Acquis Communautaire (NPAA) is a basic instrument for preparation of the Slovak Republic to the full membership in the European Union. Within the NPAA, the pre-accession assistance tool ISPA is expected to support large infrastructure projects (over EUR 5 million) devoted primarily to drinking water, wastewater management, waste management and air pollution.

Year 2003 has been the last year for the preparation of ISPA applications in Slovakia. After the accession to the EU this programme will continue under the Cohesion Fund, which is now available for Ireland, Greece, Spain and Portugal. In fact, ISPA has been closely based on Cohesion Fund rules and was aimed as a "training tool" for candidate countries to apply for assistance in large scale infrastructure projects in the transportation and environment sectors. While under ISPA the ceiling of the intervention rate has been 75%, it will be as high as 85% under the Cohesion Fund. Consequently, these instruments represent a major tool in the upgrade of water and sewerage facilities in Slovakia and other candidate countries for accession to the EU.

There have been a number of successful ISPA applications developed in Slovakia since 2000. Below, a brief description of the largest ISPA 2000 project, Banská Bystrica Wastewater Disposal System, is detailed, followed by a short description of the first Cohesion Fund application in the environment sector prepared in 2003 and entitled Water Supply and Sewerage of Horné Kysuce. Both applications represent integral solutions to water and sewerage problems of relatively large agglomerations and have been developed in line with the principles of integrated water resources protection and wastewater management.

7.1 BANSKÁ BYSTRICA ISPA PROJECT [3]

The City of Banská Bystrica is situated in the heart of Slovakia, about 200 km from Bratislava. It has approximately 90,000 inhabitants, including nearby villages. The city is served by a combined sewer system. The nearby villages of Malachov, Tajov, Badin, Selce, Kynceľová and Nemce do not have proper sewerage facilities. Thus, storm sewers to which building connections have been connected during the years of operation discharge raw sewage into the local streams without any treatment. In addition, there are 32 CSOs in the sewer system discharging wastewater to the Hron River and other local receiving streams. As a consequence of the incomplete sewer system, CSO pollution and inefficient treatment of wastewaters, Banská Bystrica has become the largest single polluter in the Hron River Basin. To overcome this unfavourable situation an ISPA application has been prepared in 2000.

The main problems in Banská Bystrica can be summarised as follows: insufficient capacity of interceptor A and other main or trunk sewers; large amounts of infiltration water in sewers due to the intrusion of groundwater; interconnection between sewers and local streams, resulting in large amounts of surface water inflow; frequent flooding of parts of the city caused by the insufficient capacity of the sewers; incomplete sewer sections leading to the pollution of streams and the Hron River; and, inadequate treatment of domestic and industrial wastewaters at the WWTP.

Given the above list of problems, it has been decided, that after a steady flow redesign of the entire sewer system, a flooding check will be carried out. The results of the flooding checks, except for some short sections of street sewers, confirmed the correctness of the design. As seen in Figure 4, sewer A was correctly designed; the model reported no surface flooding (MOUSE and HydroWorks have been used for sewer modelling). Surcharging occurred only in a few sections of this sewer but water levels in the surcharged sections were at acceptable depths (about 2.0 m below the surface elevation).

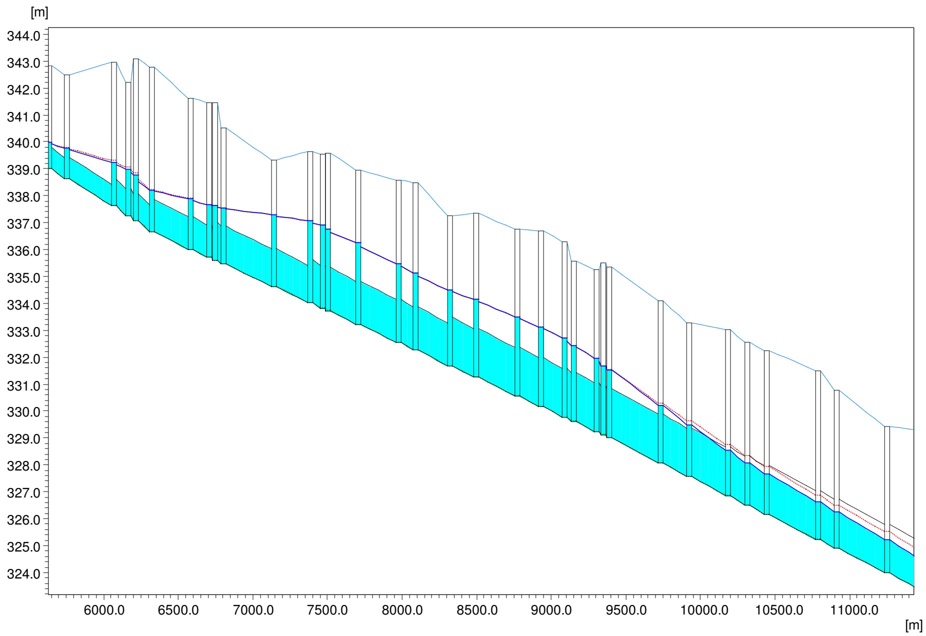

Figure 4. Result of dynamic modelling of the sewer system in Banská Bystrica

As a next step, the modelling of CSOs was carried out using MOUSE SAMBA and resulted in a proposal for modification/closure of some CSOs. As a part of the integrated approach, river modelling is currently being prepared by the Department of Sanitary Engineering of STU Bratislava. However, we envisage that this task, due to insufficient data on the pollution of the Hron River, will take a longer time to complete compared to the modelling of the sewer system.

7.2 HORNÉ KYSUCE COHESION FUND APPLICATION [4]

Development of this project has resulted from an integrated river basin management approach adopted by the Northern Slovak Water and Sewerage Company Žilina (SVS, a.s.). SVS, a.s. administers 5 major investment projects (4 ISPA projects, plus a Phare CBC project) in the upper Váh River basin (see Figure 5), all under the umbrella of SVS, a.s.

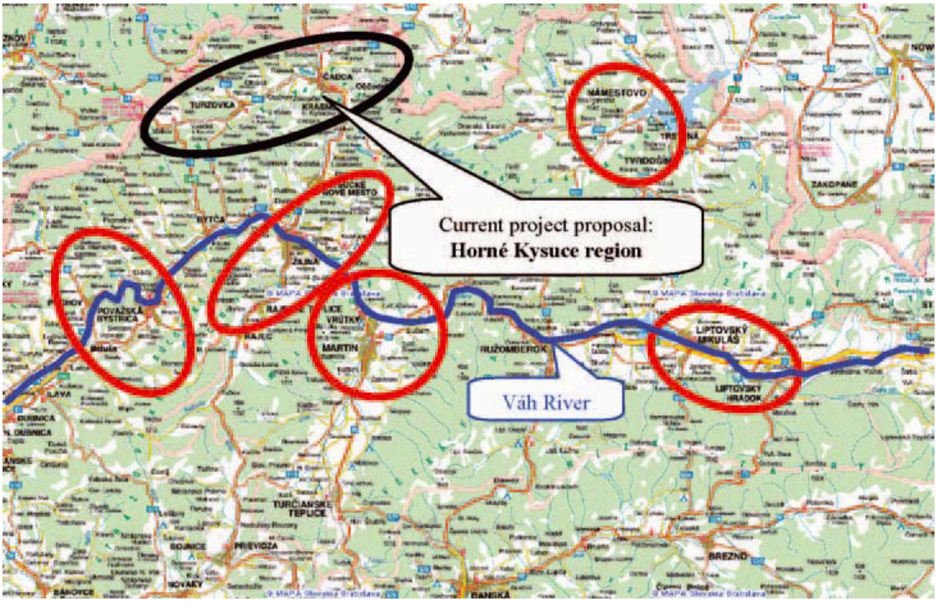

Figure 5. EU funded projects of SVS, a.s.

The project area of the Horné Kysuce application is located in Northern Slovakia and includes 14 municipalities, all with 2,000 or more inhabitants and a total population of 70,000. Out of this number 67% (46,000 inhabitants) is currently connected to piped water systems. There are frequent periods of water shortage during dry weather conditions in the whole project area. Only some 32% of the total population (22,000 inhabitants) is served by public sewerage, the rest is connected to cesspits, septage tanks and a few local WWTPs. The objectives of the project are therefore connection of the majority inhabitants to public water supply and sewerage networks, and reduction of the river pollution.

As a result of a comprehensive study of existing water resources and computation of a detailed water balance, it has been proposed to connect the region to the regional water supply system SKV Nová Bystrica (see Figure 6). On one hand, sufficient local sources are not available (flysh zone) in the requested amount, and those available are unsafe to use due to their significant seasonal variations.

Moreover, surface water resources (although declared as sources for drinking water withdrawal) are heavily polluted by bacterial contamination (described by *E.coli*).

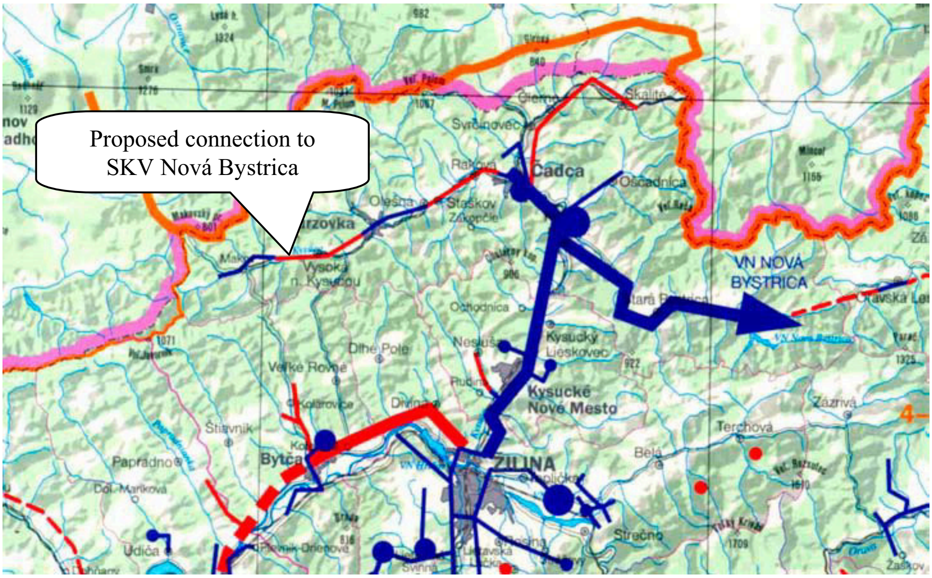

Figure 6. Scheme of SKV Nová Bystrica (existing and proposed networks)

The resulting design proposes the continuing use of local sources with sufficient quantity/quality and the coverage of the rest of inhabitants from SKV Nová Bystrica (see Table 3). After completion of the measure in 2009, 84% of inhabitants will be supplied with good quality drinking water.

Table 3. Proposed solution for drinking water supply

Parameter	Unit	Max. demand	Coverage from SKV	Coverage from local sources
Water demand	l/s	219.6	**183.5**	36.1
	m^3/d	18,969	**18,850**	3,119
	%	100 %	**84 %**	16 %

As the current state in sewerage is more serious compared to that of water supply, even more rigorous study was carried out in this field. Alternative solutions of wastewater collection and treatment have been examined, taking into consideration technical aspects, capital and operational costs as well as river pollution aspects. It was concluded that two central WWTPs will be constructed (one of them already exists, but needs to be extended). It has been proven that such a solution gives the most economical solution coupled with a better spatial distribution of the effluent pollution discharge into the local river.

Table 4. Proposed solution for wastewater collection and treatment

		Year 2001	Year 2009	Year 2030
Inhabitants				
Total number of inhabitants	No.	68 756	69 752	72 437
Connected to sewer systems	No.	22 002	49 969	61 609
With cesspit waste transport to WWTP	No.	900	1 774	6 365
Total inhabitants to WWTP	**No.**	**22 902**	**51 743**	**67 973**
	%	**33%**	**74%**	**94%**
Inhabitants not connected to WWTP	No.	45 854	18 009	4 464

As seen from Table 4, the proportion of inhabitants connected to the public sewer systems in 2009 will raise to as high as 74%.

8. Conclusion

As described in the paper, in the future, the Slovak state water management policy has to be based on well defined water management objectives, which *inter alia* include:

- Provision of sufficient volume and quality of drinking water for inhabitants and other consumers in concert with the Directive 98/83/EEC;
- Provision of collection and adequate treatment of wastewater in line with the Directive 93/271/EEC and especially with respect to the polluter pays principle;
- Achievement of a high level of environmental protection within the context of sustainable development; and,
- Achievement of an adequate level of flood prevention.

The ultimate goal of Slovakia is to harmonise water management policy with the requirements of the European Water Framework Directive 2000/60/EC.

9. References

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URBAN WATER AND CLIMATE CHANGE

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1. Is the global climate really changing?

There is no chance to deny that the world's climate is changing, at least in the last decades. This is clearly perceived by the common sense, at least in the European countries, and especially in the Mediterranean area. The question is still open, and to give a satisfactory answer is a very difficult task, because there are several phenomena not yet completely understood and many points still need to be investigated. There are two possible interpretations concerning, respectively, a persistent trend with sharp characteristics, and the occurrence of long period oscillations with very high extreme values.

Recording unusual precipitation shortage and temperature increase, particularly in the last years, justifies saying that the actual meteorological pattern is somewhat different from that we were accustomed to just a few years ago. The effects of such a new climate are felt in many ordinary aspects of the daily life, and the people who have responsibility to run the governmental apparatus are facing unexpected problems, in order to mitigate the trouble affecting all the population levels.

In such a complex framework it becomes reasonable and opportune to try and see whether the climate change does affect also the usual problems of the urban drainage and what sort of considerations should the administration take, and also to adopt preventive measures. These considerations are quite general and should be tackled with the most appropriate care, even though not all the urban agglomerations are showing so far an alarming status.

2. The risk of drought

To say that the rainfall has decreased and the air temperature has increased does not allow saying immediately that we are going toward a drier future, in which more and more regions will experience the conditions typical only for some areas characterised by water shortage. This reasoning entails to take into due consideration the concept of drought and its substantive characteristics.

Drought is in fact one of the main subjects of today's scientific speculation and several studies have been carried out so far, while other investigations are now in progress, with respect to significant areas to keep under control and with the application of many multidisciplinary tools in an attempt to interpret the recorded

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data. To quote just a few examples from the abundant scientific literature, Rossi et al. [1] and Rossi [2] have recently examined the characteristics of drought, as a multi-facet phenomenon connected to meteorology and hydrology. Drought can be defined by means of appropriate indices that take into account the effect of water scarcity, particularly in the agricultural domain [3]. A permanent drought monitoring structure has been proposed for the Mediterranean countries [4].

Concerning the urban drainage, a drought is to be considered in relation to the shortage of rainfall occurring in the occupied area. On the other hand, a shortage of rainfall can occur also without the other peculiarities that characterise the drought. In this context, it seams more appropriate to restrict the attention to the meteorological pattern, leaving the drought aspects to more specific problems.

This is also justified considering that the urban drainage is affected directly by the presence of rainwater, to be collected and removed, and very few implications exist with the various phenomena that govern the precipitation.

The change of precipitation pattern is observed in many places, regardless the normal environmental conditions.

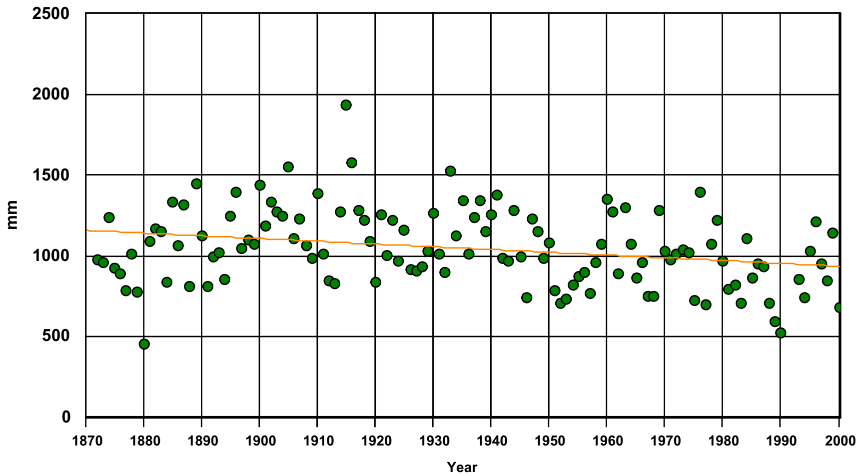

Fig. 1. Total annual precipitation at Caserta (central Italy), for a long recording period.

Fig. 1 refers to a very long precipitation record at a location typical of the Mediterranean area. After analysing the annual values by means of a simple linear regression, a downward trend is clearly evident. If the trend will continue in the immediate future, the probability of having less water to be removed will increase.

Fig. 2 refers to another climatic region, in the core of the Alps, which is normally considered a wet area. Also in this case, a simple linear regression of the recorded data shows that the total number of days with no precipitation tends to decrease.

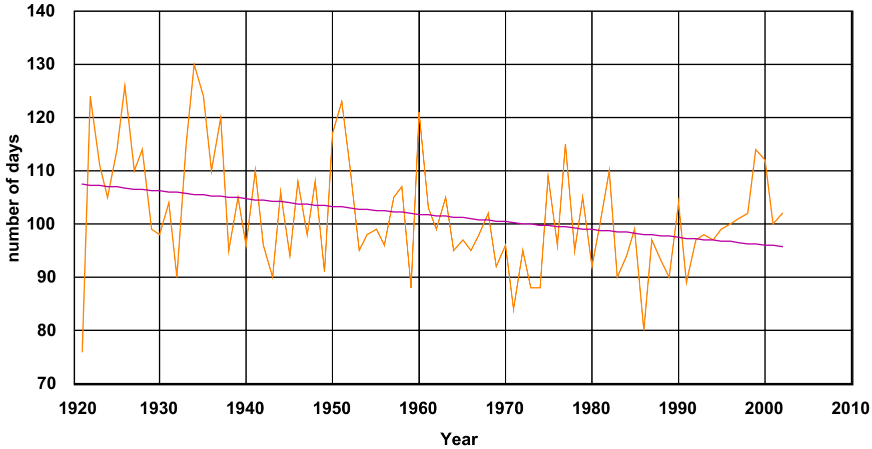

Fig. 2. Number of rainy days per year at Sesto, in the alpine area.

This situation is further examined in Fig. 3, in which the reduction of precipitation in a winter month is emphasised. In this area the last years have been characterised, in particular, by a very remarkable shortage of snow.

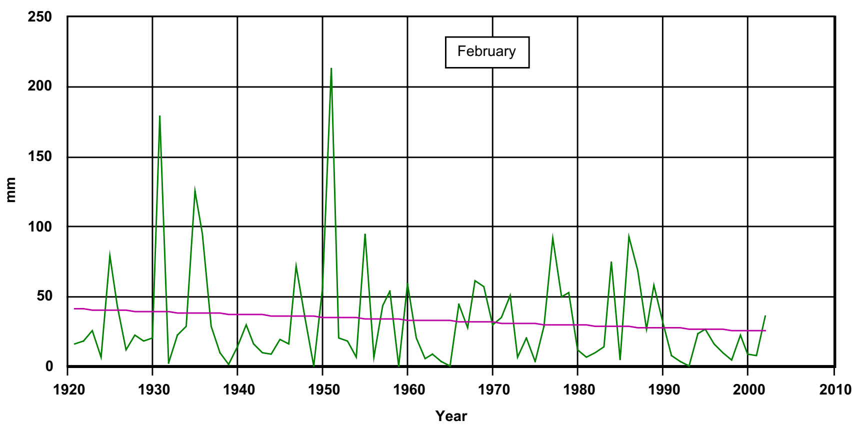

Fig. 3. Precipitation in a winter month at Sesto.

There are also indications of unusually high values of the air temperature, not only during the summer months.

As a general consideration, the following remarks can be drawn:

- a climatic change is ascertained, particularly in the warmer season, both in terms of lower precipitation and lesser number of rainy days; and,
- the temperature increases.

Such a reality cannot be neglected in the urban context, where the removal of rainwater is one of the main problems for the responsible authorities. This problem is strictly connected to the people's health and to the environmental problems as well. In the following pages it will be examined in detail, in an attempt to seek what can be its real impact on the correct design and operation of an urban drainage network.

3. The increase of storm events

Clearly the change of climate is not only detectable in terms of precipitation shortage and temperature increase.

The current situation is now studded with an increasing frequency of events of high intensity and short duration rainfall. They cause floods in rivers and in the urban context they exacerbate the problem of rainwater removal by means of the existing sewage networks [5].

The increase of similar events seems to confirm the overall assumption that the total amount of water falling on the globe is constant over a long time interval, and only its time and space distributions have changed. In other words, a long period of dry conditions is counterbalanced by short periods of very intensive rain.

The occurrence of such high intensity events is observed all over the world. A typical situation can be again that relevant to the city of Caserta, shown in Fig. 4, located in an area normally characterised by water scarcity, particularly in summer.

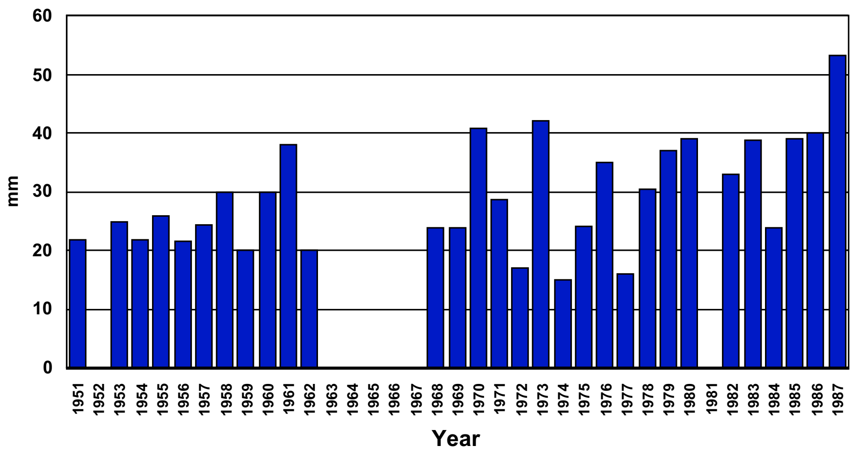

Fig. 4. Maximum hourly precipitation at Caserta

While the rain gauges denote a high number of peaks, mostly in a range shorter than the half of an hour, with intensity reaching sometimes 150 mm/h, an interesting analysis can be carried out considering the cases in which at least one rain gauge has recorded an intensity exceeding 80 mm/h. The result of the analysis, sketched in Fig. 5, confirms that the frequency of such events has clearly increased in the last period.

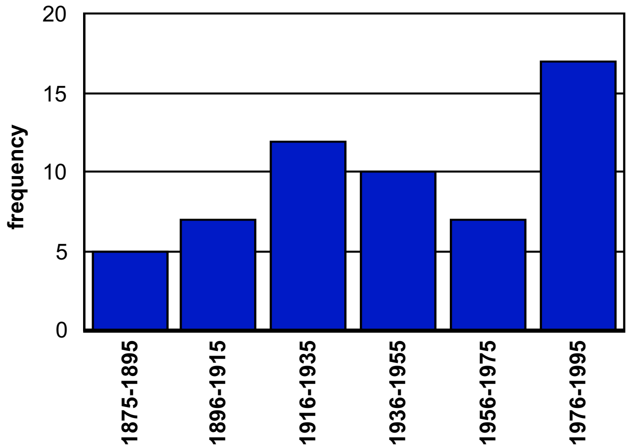

Fig. 5. Frequency of the rainfall events in which at least one of the rain gauges in Naples has recorded intensity more than mm/h.

Also as far as the peak precipitation is concerned, the new climate pattern shows that the actual meteorological characteristics have significantly changed, in comparison to those usually considered a few decades ago.

4. Dry periods and urban drainage

Usually the occurrence of a drought, or, at least, of a dry period, is not considered among the urban drainage problems. Both in design and operation, the main interest is concentrated on the high precipitation and, consequently, on the prevention and mitigation of floods. This can be justified in principle, as the urban drainage network is conceived as the tool for removing the rainwater fallen on the ground. If there is no water to remove, there should be no problems.

Is this entirely correct? Is there a risk that a drainage network can suffer for having no water?

When operating, the drainage pipes should convey the water coming from pavements, roads and roofs, where it had fallen before in form of rain. Yet the urban context is really very complex, and it is always possible that a certain quantity of water of other than meteorological origin can reach the pipe. It can happen through the openings specifically designed for such purpose, but more frequently, it is due to accidental leakage. In this respect, the technical literature deals with the so-called "dry weather flow", to be considered in terms of:

- drainage of groundwater
- road washing
- spills from industries and handicraft activities
- car washing, and
- gardening and plant irrigation.

The presence of dry-weather flow is enhanced in the case of combined sewer systems, commonly used in the Mediterranean countries, and particularly with no exception in Italy. In this case the sewage pipes have to convey also the wastewater coming from households.

The pipe's layout and hydraulic characteristics affect the dry-weather flow. In cities in flat areas, sewers have to be regularly flushed by automatic devices to operate well in dry weather. Another set of problems arises if the water quality is considered. The relatively small dry-weather flow carries normally high concentration of solutes and suspended solids, which can settle and accumulate on the pipe's bottom and stay there as long as the flow is not strong enough to resuspend and remove them. This is done by rainwater, when it enters sewers as a substantial flow generated by an appreciable precipitation event and provides sewer cleansing.

The drainage network is always designed in relation to the highest flow to be conveyed. This is achieved in terms of slope, roughness and hydraulic radius. Implicitly, any flow smaller than the maximum can move easily, and this could be valid also for the dry weather conditions.

Nevertheless, a small quantity of water on the bottom of a large pipe has a small hydraulic radius and, consequently, its motion is hampered and the settling of suspended material enhanced. The sediment deposition on the bottom eventually results in an increased roughness that further reduces the water velocity. After a long period of dry weather, the pipes have reduced capability of conveying great quantity of water, eventually even with respect to the relevant design flow.

Long periods without water collecting and removing bottom sludge entails also some important aspects relevant to the appurtenances of the sewer network. Road and curb inlets, manholes, culverts and weirs become the sites of solid deposition and accumulation. Such a phenomenon is aggravated by the road works carried out on the street surface, or by the accidental spills of solid waste.

Deposition and accumulation of solid material is caused by insufficient velocity of water during dry weather, and it is particularly noticeable in pipes with low slopes. To maintain the pipe conveyance capacity, it is a common practice to flush the pipe by injecting periodically a proper amount of water by means of automatic devices. The flushing water can help also to "wash" the pipe's bottom and remove the solid material and organic matter, accumulated when the same pipe was more or less empty. Keeping always these devices in good working conditions entails constant attention and costly interventions. Nevertheless, the flush water cannot remove completely the crust that can develop on the inner surface of the sewer pipes.

The water quality aspects of the problem discussed deserve a particular attention, as the solid material, either accumulated or crusted, contains very often pollutants and toxic substances, like those due to the emission of road traffic. The presence of bacteria and other living organisms can activate biological processes, very sensitive to the air temperature. Particularly in summer, there is a risk of odour emission, unacceptable in the urban area. As an extreme consequence, contagious diseases can rise and propagate in the surrounding places.

The ultimate effect of such a complex plight manifests itself at the occurrence of a substantial rainfall, when the generated flow has, first of all, to remove the solid material from the bottom and "wash" the pipe. The so called "first flush flow" becomes therefore extremely polluted and its discharge into a receiving water body can cause unexpected environmental impacts.

As an overall consideration, neglecting the possible occurrence of a long dry period is not a sound attitude in the management of an urban network designed to protect a community from rainfall events.

To complete the picture, it should be remembered that a shortage of available water imposes also serious problems of meeting the demand of several uses, which are essential for correct operation of all urban activities, and, from the environmental view, has now implication with the quality of air, water and soil in the urban aggregation. Finally, the shortage of water in a receiving stream prevents the removal of pollutants discharged from a town.

Obviously, the outcomes described above can be observed whenever the drainage network is without water for a long time, but it is quite clear that the new climatic pattern will increase their frequency and enhance their effects.

5. The effect of storms

An urban drainage network is always designed for the removal of the flow caused by rain. To this purpose, the best hydrological information is necessary and several computation methods are available in the technical literature, with their validity confirmed by countless examples of systems working correctly all over the world.

The current procedures take into account the extreme precipitation values, in terms of high intensity. However, the increased frequency of peaks, experienced during the recent years, and the greater recorded values as well, were not included in the hydrological information on which the existing network had been designed. The probability of failures has therefore increased, with more expected cases in which the drainage networks will not be able to remove the water generated by the highest precipitation events. In practice, more frequent inundation events are expected in the areas served by the drainage networks.

If this hydrological trend will be confirmed in the future, more and more adequate measures are necessary to improve the efficiency of the drainage networks. In the urban context the effects are now particularly serious [6]. The occurrence of s sudden flood is a risk involving the safety of inhabitants, the road traffic and the preservation of historical and artistic sites.

6. Intervention and measures

In view of the negative aspects mentioned above, it seems quite correct to search for some actions to take, both for the existing structures and for those to be designed. As a general rule, the size of the pipes should be increased, as well as that of the various components of the sewer system. Intervention during the design stage will provide more choices. Vice versa, for the existing networks any attempt to improve the efficiency can entail demolitions, excavations and rebuilding, with high costs and inconvenience for the affected people.

A surrogate solution, which is not necessarily an alternative to rehabilitation and rebuilding, can be to convince the people to buy appropriate insurance. This solution is already favoured in many countries, where the insurance companies have worked out suitable policies for this purpose.

In any case, a very thorough analysis is necessary, in order to ascertain the level of risk that characterises the failures. Several tools are available, which also allow evaluating the inherent economic impact. The risk can be also illustrated by means of graphical procedures, like the maps showing the areas prone to inundation in the case of certain hydrological events.

7. Coping with water shortage

For specific cases of persistent dry weather and increasing frequency of high flows, a more detailed analysis of the feasible measures should be beneficial. Concerning the occurrence of long dry periods, the principal recommendation for an existing network is to perform accurate maintenance, and checking the proper working conditions of various system components, like pipes, inlets, culverts and weirs. Maintenance must be done very frequently, by well trained people, who should have the best knowledge of the network and its main risks of failure.

Vice versa, when designing a new network the attention should focus on adopting the hydraulic criteria, which can assure the best conveyance of dry weather flow, avoiding deposition of suspended material and formation of obstructions. Specific considerations include:

- Select the pipe cross-section in such a way as to obtain the highest velocity compatible with the smallest degree of filling (e.g., egg-shaped cross-section);
- Choose the pipe material and the construction procedure offering the least relative roughness;
- Increase the number of manholes along the sewer for easy inspection;
- Provide the sewers, especially those with low slopes, with frequent flushing of suitable discharge;
- Prevent and mitigate the occurrence of odour, fermentation and growth of dangerous bacteria; and,
- Arrange suitable points for pollutant measurement and control.

Normally these actions should impact much on the construction costs, but would increase operational costs, by requiring extra manpower. This should be considered when planning the operation procedures.

8. Coping with high flows

The measures concerning the increased occurrence of high flows are quite different. Also in this case a proper maintenance, aimed at reducing the risk of malfunctioning, is essential. Frequent inspections and interventions can confirm that the sewer and other components are functional, free of obstructions, and free of risk of water overflow on the ground. This maintenance activity is obviously associated with that previously described for dry weather conditions.

More difficult appear to be the measures needed to improve the efficiency of an existing network. They have to be decided after an accurate analysis of the expected costs.

Replacing the existing sewers with new ones is the most effective solution, after an appropriate hydraulic calculation, taking into account the new peak flow to be conveyed. Improvement can be also done on appurtenances, particularly the road inlets, to facilitate the collection of rainwater from the ground and its transfer into the sewers.

Another option to be considered is the construction of storage tanks, preferably underground, which could retain a portion of the peak flow. The retained water can be released afterward, once the sewers have discharged the quantity of water they were able to receive. As an alternative, the stored water can be used for some non-potable purpose after the rainfall cessation. It is a solution currently applied in some countries, but the possibility of its transfer to Europe still needs to be thoroughly examined.

When planning a new drainage network the attention must be paid to choosing the correct hydrologic information, with the significant extreme values, allowing to calculate the maximum flow. Moreover, the design of a new network should incorporate all the possible improvements described earlier. A consideration of emergency devices is also a good recommendation, should the designed network not work properly.

9. Conclusions

In the past, according to a consolidated policy of managing urban activities, the extreme hydrological events were considered mostly with respect to their emergency nature. Particularly in the case of floods, more attention was given to remedies than to prediction and mitigation of the effects.

In the case of drought or persistent dry weather, many points still need appropriate investigation. A shortage of water not only requires the responsible authority to provide other water supplies to the citizens, but also to deal with predominant sanitary and environmental aspects that were normally considered of lower priority. To meet correctly the requirements for which an urban drainage network is designed and constructed, an accurate hydrological analysis is necessary. The calculation of the maximum flow to be conveyed requires the evaluation of some extreme events that probably were not known and conceivable a few decades ago, but are now more and more frequent. They follow a climate trend that is confirmed by the last records and seems to continue in the next future.

If the removal of the maximum flow is the main objective for constructing an urban drainage network, the occurrence of long dry periods, which is characteristic more and more for the current hydrologic regime, must also be taken into consideration. Even though the effects appear less dangerous at first glance, the shortage of rainfall can give rise to some negative outcomes, which cannot be neglected for the correct operation of the sewer systems.

A thorough maintenance is essential for keeping the drainage network in good running order, with adequate measures in terms of manpower and financial resources.

As an ultimate consideration, it is very opportune to recommend that the people responsible for designing and running the network should perform an accurate monitoring activity in order to provide the information, in terms of hydrology and water quality, necessary for the correct functioning of the network.

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ENVIRONMENTAL PROTECTION TECHNOLOGIES FOR SUSTAINABLE DEVELOPMENT

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1. Sustainable development and environmental protection

The quality of our environment is a matter of eminent concern for all of us, especially for future generations. It is increasingly felt that the time has come for some drastic changes in the way our environment is protected from pollution, i.e., how to maintain a high diversity of life, at local and global levels, and how to prevent the exhaustion of resources. Our society urgently needs a sustainable lifestyle, which also adheres to the environmental protection technologies adopted [1]. This implies a holistic, multidisciplinary approach to current global problems, such as overpopulation, malnutrition, desertification, water quality deterioration, and so on. From the ecological point of view, one knows that uncontrolled exponential growth should be prevented in an ecosystem if the population wants to colonise that ecosystem in a sustainable way. Harmony and balance are needed around the carrier capacity of an ecosystem, and one can question if sustainability and growth can really co-exist. Sustainability and exponential growth certainly cannot. With respect to human society this not only applies to the prevention of the exhaustion of resources, but also to social justice; that is, the prevention of extreme prosperity of a few at the expense of hopeless poverty of the vast majority.

The concept of sustainability is certainly not new. However, it is difficult to be understood and implemented worldwide. The lack of parameters to quantify sustainability contributes to the vagueness of the concept. This leads to ambiguously defined targets or actions proposed by politicians and/or policy makers, although this can be on purpose on some occasions. Even when international committees are involved, such as the well-known Bruntland Commission, the definition given to the concept is so open that it is easy for people, institutions and governments to avoid taking proper measures. For instance, when governments come up with extremely stringent standards for protecting the aquatic environment from pollution, the question arises, 'What sense does it make to pursue a paradisiacal natural environment in a single country or region, when at the same time little if any money or technology is made available to contribute to the highly needed environmental improvement in less prosperous countries?' This is a type of environmental tunnel vision which presumably mainly serves to give the citizens the impression that politicians are concerned about having a clean environment. But in fact it mainly

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serves the short-term objectives, which have little to do with sustainable and robust environmental protection. As a result, developments frequently move in the opposite direction to that which was originally planned.

2. Sustainability in industrial wastewater treatment

Adequate availability of energy and resources is a prerequisite to achieve socio-economic development that is required to improve the quality of life of present and future generations. Other important parameters for an intelligent use of material and energy sources are affordable costs, social acceptance and opportunities for local employment and new industrial activities. The provisions and use of resources, water and energy should in itself be consistent with pursuit of sustainability. Efficiency is also required to reduce the production of waste, posing new challenges for the ecological design of industrial processes [2].

By framing environmental degradation as a matter of inefficiency, technology can be seen as something, which creates, at least potentially, new opportunities and new arrangements that counteract or even prevent pollution. To date, dematerialisation of the economy and the closing of resource and material cycles are popular policy concepts developed with the goal of managing resource and material flows. However, the scientific basis for these concepts is still relatively weak. Various inconsistencies can be identified in national and international environmental and economic policies. Moreover, the ways and means for the dematerialisation of the economy and the closing of resource and material cycles are still largely unknown [3].

Nevertheless, the scientific and technological challenges in the field of closing resource and water cycles are manifold. In general, closing these cycles will involve modifications of the production process. Process optimisation makes use of dedicated analytical tools, such as water pinch analysis [4], which allows determining the most efficient water utilisation schemes. The classical "end-of-pipe" treatment of waste streams is replaced by "in-line treatment" processes. Although both approaches use the same unit operation processes, e.g., membrane filtration or biological degradation, the applicability of the unit processes for "in-line treatment" needs to be revisited, as changes in water quality, e.g., higher concentrations of organics or salts, can make them inadequate.

Industry expects that integrated, preventive biotechnological applications will become more important in the coming years [5]. One way of using biotechnology in order to prevent pollution is replacing a chemical process by a biocatalytic one, thus by adopting "green chemistry". Biocatalytic processes offer promising routes for more environmentally sound processes. Twenty years ago, one of the great promises of modern biotechnology was that it could make industry greener, i.e., by making use of bio-catalysts. To date, there are still only few biocatalysts in industrial use [6]. The basic idea is that the pollution that would occur by using the chemical process does not occur by using the bio-catalytic one. This does not mean that the biocatalytical process is always superior to the chemical one. However, biocatalysts offer good possibilities for a more eco-friendly production.

Adoption of clean technology goes one step further than green chemistry and re-evaluates the complete production process. In clean technology, the life cycle of a product is considered and it is attempted to minimise the use of resources as well as the amount of emissions during the life span of a product [3]. The use of a clean-up technology reduces the environmental impact, but also increases the economic costs. By selecting clean technologies, i.e., a completely different production process, the environmental impact can be reduced at lower economic costs.

In industrial ecology, material streams are kept as long as possible within the industrial ecosystem. This is achieved by reuse, recycling or by the use of waste as a resource for a different industrial application. This was, for example, achieved in the currently best known example, the Kalundborg case. The leading device in industrial ecology is "to do more with less". In this approach, not only the impact on the environment by the design of a product is considered, but also the recycling logistics needs to be addressed (Fig. 1). Examples are the design for environment (DFE) and design for disassembly (DFD) in car manufacturing. Thus, industrial processes move towards the engineering of the entire life cycle of a product [7].

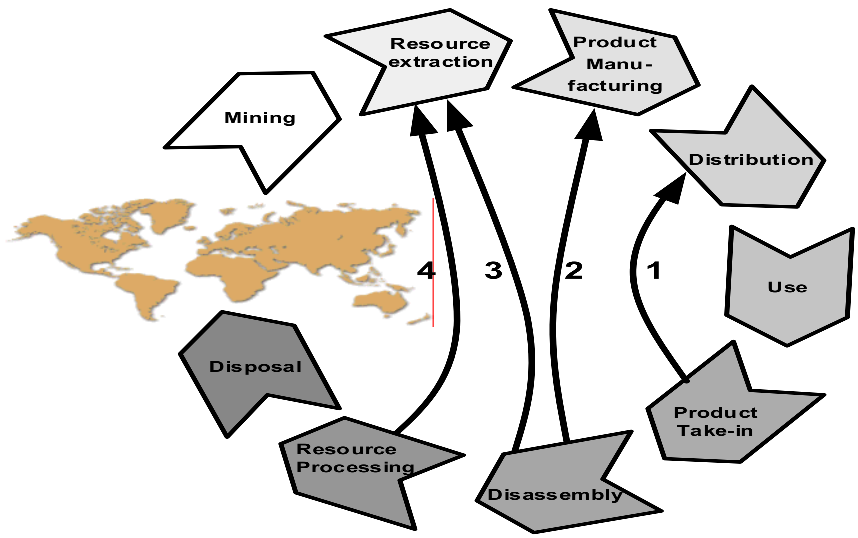

Fig. 1. Reuse and recycling opportunities in the production chain (After: Bettac et al., [8]. Reuse: 1. Direct reuse, 2. Reprocessing of reusable compounds. Recycling: 3. Recycling of resources, 4. Regeneration of resources.

3. Sustainability in organic solid waste management

Uncontrolled spread of waste materials leads to health problems and environmental damage. To prevent these problems a waste management infrastructure has been set up that collects and disposes of the waste. Current waste management schemes are generally based on a hierarchy of three principles: waste prevention, recycling/reuse and final disposal.

Final disposal is the least desirable option as it causes massive emissions to the atmosphere, water bodies and the subsoil. The emission of methane to the atmosphere is an important source of greenhouse gasses. Organic waste therefore gets a lot of attention in waste management, which for Europe can be illustrated by the issue of the Landfill Directive (99/31/EC) and the Sewage Sludge Directive (86/278/EEC).

Proper treatment of organic waste may however turn this burden into an asset. Especially biological treatment may help in acquiring more effective resource management and sustainable development. The following advantages may be listed:

- The greenhouse effect is tackled as methane emissions from landfills are prevented.
- Soil quality can be restored or enhanced by the use of compost in agriculture.
- Compost may replace peat in horticulture and home gardening, reducing greenhouse emissions and wetland exploitation.
- Anaerobic digestion has the additional benefit of producing biogas that may be used as a fuel.
- Pesticide use can be reduced by proper use of the disease suppressive properties of compost.

As a result of this linkage of the field of waste management to the societal production system at large, there is a need for involvement of many different disciplines, not only natural sciences but also economic, political and management sciences, product specialists, etc. Although most people involved will agree with such a statement, in practice we still see a lot of fragmentation. As an example there is still insufficient exchange between agronomists working with compost and technologists working with the composting process.

4. Sustainability in domestic wastewater treatment

4.1. CENTRALISED SANITATION AND SUSTAINABILITY

Of all the wastewater in the world, 95 per cent is released to the environment without treatment [9]. In 1997, three billion people on earth lacked adequate sanitation. If sanitation provisions continue to be installed at the current rate, up to 5.5 billion people will be without sanitation by the year 2035, many of whom will be living in crowded urban settlements [9]). As a consequence of this lack of sanitation, 3.3 million people die annually from diarrhoeal diseases, out of 3.5 billion infected. In Africa alone, 80 million people are at risk from cholera, and the 16 million cases of typhoid infections each year are a result of lack of adequate sanitation and clean drinking water [10]. Although inadequate sanitation is less of a problem in European countries, regular epidemic outbreaks (for example, of *Cryptosporidium*, *Gardia* and *Legionella* and even cholera) indicate that developed countries also face problems of improper sanitation.

One of the main reasons for this situation is the high cost of current water-borne sanitation techniques and methods. It is obvious that the established sanitary engineering world in the public sanitation sector emphasises the implementation of very expensive (both in investments and operation) high-tech centralised systems. This applies to the processing and distribution of drinking water and to the collection, transport and treatment of solid waste and wastewater. A huge number of such centralised urban sanitation (CUS) systems have been developed and implemented in the last century, especially in the industrialised world [11]. Huge investments have been made to install the sewerage systems required, and the maintenance of these systems is also expensive [12]. According to Grau [13], countries with an average annual per capita gross national product (GNP) of below US$1,000 not only lack the resources to construct treatment plants, but also cannot maintain them, even if these plants were constructed free of charge. Moreover, as the lifespan of sewers is only in the order of 50-70 years, these investments have to be repeated periodically.

CUS systems are based on the collection and transport of wastewater via an extended sewer system to centralised treatment systems. These systems use clean water (mainly tap water) as the transport medium of domestic wastes which are frequently relatively concentrated as is the case, for example, of faeces. Moreover, very little, if any, recovery of useful by-products such as fertilisers (Nitrogen, Phosphorus and Potassium) is achieved. On the contrary, huge amounts of poorly stabilised and polluted sludges are generated which have to be disposed of because they are not acceptable for agricultural reuse. Thus, the CUS approach is far from sustainable. The proper functioning of CUS systems depends on energy supply, computer hardware, and so on, making them vulnerable to theft, sabotage and military attack in poor and politically unstable countries.

4.2. DECENTRALISED SANITATION AND SUSTAINABILITY

One can not ignore the role that modern CUS systems have played in the efficient protection of the environment over the last century and the great increase in public comfort they have offered to the industrial Western world. However, many aspects of the CUS concept conflict with sustainable sanitation (see Table 1). Enormous amounts of clean water are wasted by using it as a transport medium. Since the waste is highly diluted in this way, expensive, energy-consuming and technically complex wastewater treatment technologies have to be applied. Thus, CUS systems constitute a heavy financial burden on the society, particularly in less prosperous countries. Moreover, many sewerage systems cannot cope with stormwater and, during periods of heavy rainfall, untreated wastewater is released into the environment via sewer overflows. CUS concepts are not sustainable because resources are consumed and – except the treated water – not recovered.

Table 1. Criteria for sustainable sanitation

Little, if any, dilution of high strength domestic (and industrial) residues with clean water
Maximisation of recovery and reuse of treated water and by-products, e.g. for irrigation, fertilisation and soil conditioning
Application of efficient, robust and reliable wastewater collection and transport systems and treatment technologies, which require few resources and which have a long lifetime

The lack of sustainability of the CUS concept becomes obvious in the case of diffuse pollution. At present, full coverage with sanitation and treatment is achieved only in rich countries, serving only 6% of the world's population. No matter how high the levels of industrialisation and modernisation are, there is still a major part of the population living in the rural countryside. Even in very industrialised regions, such as Western Europe, the US and Japan, the percentage of the population living in rural areas can still be up to 20-40% [14]. Human activities in these rural areas, i.e., isolated resorts and communities, hotels and camping sites, generate diffuse pollution by wastewater and solid wastes. A variety of land use practices, such as farming, timber harvesting, construction, mining and land disposal also contribute to diffuse pollution [15].

The generally accepted concept that water can be obtained from nature in any quantities by the use of suitable technology has strongly contributed to this situation [9]. The question of how much water we really need and what quality it should posses, has not been quantified until recently. Water of the highest quality is needed only for drinking and cooking, making up about 5% of current total water consumption. However, at present, all water delivered is of the same quality since there is only one water distribution network. Furthermore, all delivered water will be contaminated if water toilets are used, so that all the water discharged from our houses after use is called 'wastewater', of which a large part is actually 'wasted water'.

Many of the drawbacks of the CUS approach can be overcome when applying decentralised sanitation concepts (see Table 2). These concepts put an emphasis on prevention (for example, little if any use of clean water for transport, but separation of concentrated and diluted wastewater in the house, and separate treatment of each (see Table 3), treatment in or near the community, application of low cost and sustainable treatment systems, recovery and reuse of useful by-products, also at or near the site (for example, water and nutrients for agricultural purposes, energy in the form of biogas for domestic purposes [16-18]). Decentralised urban sanitation (DUS) systems are in principle much less vulnerable, because their operation is independent of complex infrastructures such as energy and water supply; they are simple and robust [19].

Table 2. Some criteria for robust urban sanitation

Little dependency on complex infra-structural services, for example, power and/or water supply
High self-sufficiency in construction, operation and maintenance of systems (independent of highly specialised people/companies)
Low vulnerability to sabotage, destruction, etc.
High public participation; acceptable to all social actors
Applicable at any site and scale

Table 3. Prevention of environmental pollution problems

Complete utilisation of all possible waste resources
No pollution of water, soil or air
Finding a proper final destination for any type of residue

4.3. CENTRALISED VERSUS DECENTRALISED SANITATION

Thus far, the established sanitary engineering world remains reluctant to developments and new technologies which could lead to a more sustainable and robust alternative to the CUS concept, for example, Decentralised Sanitation and Reuse (DESAR) concepts. The statements in Table 4 illustrate the doubts the established civil engineering-oriented sanitation world has in the development and implementation of low-cost, simple and decentralised solutions for environmental protection. It should also be noted that established groups working on the so-called low-cost wastewater treatment systems, such as constructed wetlands and/or lagoons, also have prejudices against both decentralised and centralised treatment. Apparently, each specialised group sticks to their own system(s), advertises them wherever possible, for their own commercial interests, and sometimes perhaps for scientific prestige. Unfortunately, as in many other fields of human development, disagreements among specialists take place at the expense of those who really need the solution and, of course, the nature itself.

According to Harremoës [11,20], 'There is no miracle "low tech" solution in sight, because it [environmental pollution] is a social rather than a technical problem.' One can wonder if the same holds for high-tech solutions: do miracle high-tech solutions exist? The crucial point to be recognised here is that good low-tech systems can be developed and even – to some extent – are already available (Fig. 3). Undoubtedly the origin of environmental pollution is more social than technical, but one of the major reasons for current environmental pollution is the continued application of non-sustainable CUS solutions. These modern Western concepts may appear to be (at least for the time being) economically and technically achievable in the prosperous industrialised world, but they are far too expensive and complex for poor countries [21]. This, however, does not stop established consultants, contractors and scientists from attempting to implement these systems in countries that lack the financial resources and expertise to operate and maintain them.

Table 4. The evaluation of DUS as an alternative to CUS methods, from the viewpoint of the established sanitary engineering world [20]

'Local wastewater treatment is not a viable solution in cities, because the approach is either "low tech", which does not live up to established hygienic requirements and risk assessment, or it is "high tech", which suffers from energy consumption.'
'Presently available "low tech" centralised urban sanitation (CUS) approaches are not simple and not easy solutions. Consequently they are not better than available "high tech" CUS solutions.'
'The present decentralised urban sanitation (DUS) systems lack adaptability to the urban environment, manageability and control (maintenance of standards).'

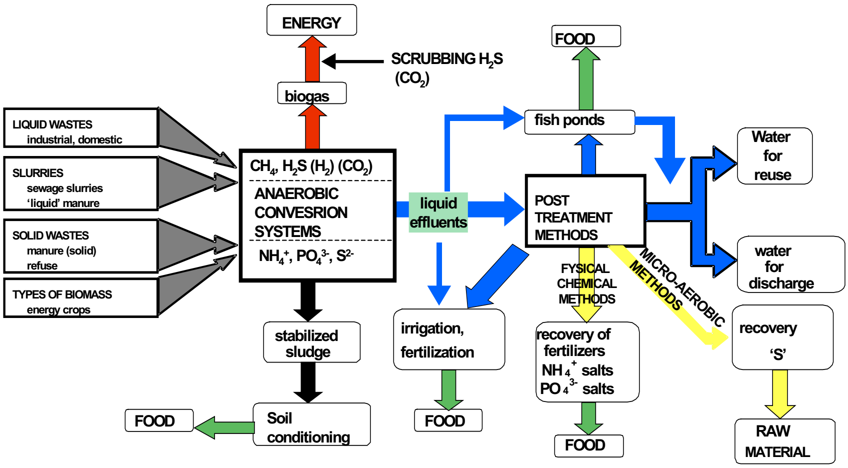

Figure 2. The potential for anaerobic conversion in the recovery of resources (water, food, new materials and energy) from waste.

In the less prosperous world there exists a growing demand for integrated decentralised sanitation systems providing opportunities to save and reuse resources, thus for DESAR-type of environmental protection solutions. It should be emphasised here that DESAR systems do not mean small scale systems. It means the redirection of the water and nutrient cycles within the community, and this can still be done using relatively large scale systems. Also, they are not low-tech either, although the core parts of the system are essentially low-tech, sustainable and robust. For the recovery of resources such as fertilisers a high-tech system and a more centralised approach will undoubtedly be needed. Each situation is unique and has its own optimal solution.

The potentials of the decentralised approach have been clearly demonstrated in the industry in recent decades with the implementation of integrated anaerobic and physical/chemical treatment, mainly in European countries. The question we face when we want to apply the decentralised approach to the sanitation sector is 'What are the best DESAR sanitation systems for the various situations prevailing in urban regions, and how to develop them?' Even when promoting the DESAR approach in the public sector, it should not be suggested that the available CUS systems should be abandoned immediately. However, attempts should be made to move step by step to more sustainable and robust sanitation systems, and to limit the extent of centralisation to reach a rational optimum, instead of implementing CUS as the only available sanitation solution. A lot remains to be done to define such a rational optimum, and it is clear that there may be several 'optima' for a specific situation, each with its own typical characteristics.

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ASSESSING EXFILTRATION FROM A SEWER BY SLUG DOSING OF A CHEMICAL TRACER (NaCl)

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1. Introduction

Water resources in urban areas comprise both natural and man-made water bodies, including surface waters and groundwater. Groundwater aquifers can be recharged from several sources, some of which may transport high concentrations of organic and inorganic pollutants. Such sources include exfiltration from leaky sewer pipes that could pose a serious threat to groundwater quality because of large extent of urban drainage systems. When the shallow unconfined and confined aquifers are connected, the transport of organic and inorganic compounds and pathogenic organisms may cause serious pollution. Consequently, the deep confined groundwater aquifers could become polluted and their water would require advanced treatment if used as a source of drinking water.

Several methods exist to evaluate the structural state of sewers. Some of these consist in direct surveying inside the sewer pipes (i.e., by closed circuit television, CCTV) and others in quantifying the exfiltration rates by the detection of wastewater markers in the groundwater [1].

The QUEST method [2] (QUantification of Exfiltration from Sewer with artificial Tracers) serves to assess directly the exfiltration from flowing sewers in dry weather. It is based on establishing a tracer mass balance for the investigated pipes. The solutions of a tracer (NaCl) are dosed in two manholes of the investigated reach, and at a downstream location, the conductivity of sewage is measured by in-line probes.

The sources of errors affecting the exfiltration rate originate from the experimental results and data analysis. In particular, they are due to: flow rate, natural wastewater conductivity, shape of the tracer signals at the measuring point, transport of tracer and general disturbances in the sewer (caused for example by turbulence or solids). To minimise the errors in experiments and data analysis, preliminary measurements of flow rate, in-sewer background conductivity and tracer transport should be carried out.

In the paper that follows, the results of application of the QUEST method to an urban sewer network in a suburban area of Rome and the importance of site-specific preliminary tests are presented.

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2. Study area and experimental methods

2.1 EXPERIMENTAL CATCHMENT

The experiments were carried out in one section of the sewer network serving the Torraccia suburb of Rome.

The sewer section studied is a part of a thirteen-year old combined sewer system built of egg-shaped concrete pipes. The investigated reaches are 4-9 m below the ground and their total length was 724 m, with a slope of 0.9 %. The tested section consists of two parts (shown in red in Fig. 1); the first one is egg-shaped with dimensions 120x180 cm and 407 m of the total length of 483 m were included in the test section, the second one is egg-shaped, 120x210 cm, and the upstream 151 m of the total length of 241 m were tested.

Since the tracer should be fully mixed at the measuring cross-section, a sufficient mixing length has to be ensured. For neutrally-buoyant tracers, the recommended mixing length for rivers is 100d - 300d (d = the channel width) [3], and application of this formula to the sewer studied yields a length of about 100 m.

The geology of the catchment area is characterised by cracked tuff and pozzolan, but the material surrounding the sewer pipes is coarse gravel used as backfill. Groundwater inundates the sewer only in wet weather, because during the dry weather the infiltrated water drains quickly through the highly permeable cracked tuff into a deeper aquifer.

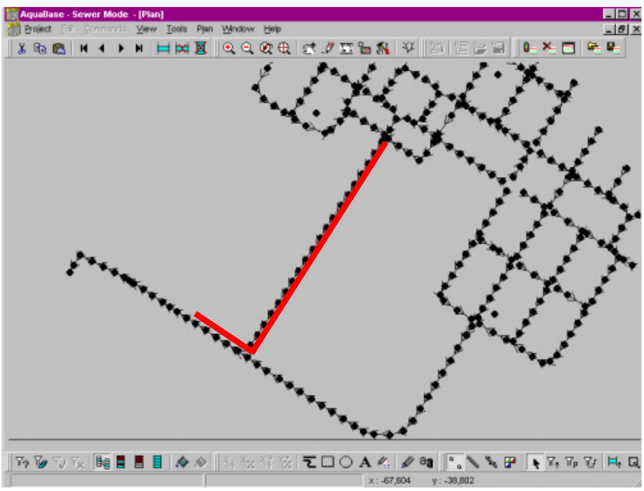

Fig. 1. Tested sewer network produced in the AquaBase software developed by DHI under the European Project APUSS (studied reaches = solid line, L = 724 m)

Although the measurements were carried out only in a part of the entire sewer system of the experimental catchment (solid line in Fig. 1), the homogeneous characteristics of the area allow extrapolation of results to the entire Torraccia sewer system.

2.2 EXPERIMENTAL MATERIALS

The equipment used consisted of one submerged probe for water level and velocity measurements (Sigma 900max) and three WTW conductivity probes (model LF 197 with sensor TetraCon 325). Conductivity data were recorded by a datalogger (GRANT SQ400) with a time resolution of one second. The chemical tracer applied was 97% pure NaCl.

3. Methods

Sewer exfiltration was measured by the QUEST method [2], which has been developed in the European Project APUSS (Assessing Infiltration and Exfiltration on the Performance of Urban Sewer Systems). In QUEST, tracer slugs of a known concentration are injected at two different manholes along the tested sewer pipe and the tracer cloud is observed at a downstream point (Fig. 2).

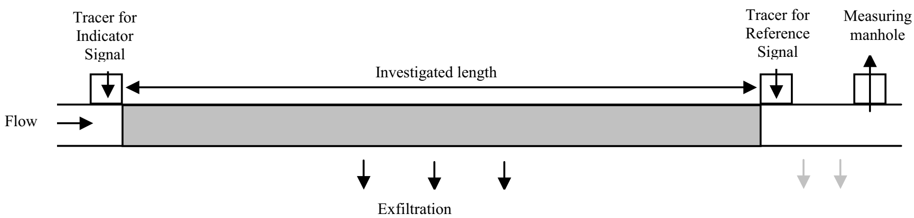

Fig. 2. Conceptual scheme of the QUEST method (modified after [2])

At the first (upstream) manhole of the investigated pipe, a known mass of the tracer is injected and as the tracer moves downstream, it is affected by exfiltration from the sewer. At the measuring manhole (Fig. 2), sewage conductivity is recorded. When the tracer cloud arrives at this section, a peak concentration is detected and called the indicator peak, because it allows evaluation of the residual tracer mass.

At the second (downstream) manhole the tracer slug dosing serves to estimate the flow rate, and the conductivity peak measured further downstream is called the reference peak. Between the second manhole and the measuring manhole, the exfiltration rate is not assessed, because it affects both the indicator and the reference signals equally.

In Fig. 3, the reference and indicator conductivity signals are shown together with the flow hydrograph.

If complete mixing occurs, the relative loss of tracer equals that of wastewater and the sewage exfiltration fraction (= exfiltrated mass/incoming mass) can be calculated as:

$$exf = 1 - \frac{M_{meas}}{M_{dosage}} \tag{1}$$

where M_{dosage} is the dosed NaCl mass [g] and the "measured" mass M_{meas} is evaluated by the following equation:

$$M_{meas} = \int_{span\_ind.} Q(t) * e * (C(t) - C_{baseline}(t))dt \quad (2)$$

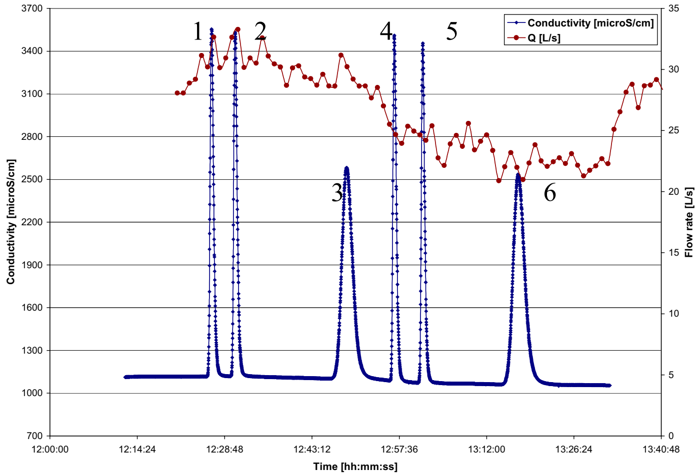

Fig. 3. Conductivity and flow rate vs. time (peaks numbered 1, 2, 4 and 5 are reference signals, peaks numbered 3 and 6 are indicator signals)

In eq. (2), *span ind.* index refers to the time interval, during which the conductivity peak of the indicator signal passes through the measuring manhole; Q(t) is the flow rate during the indicator peak passage [L/s]; C(t) is the measured indicator signal conductivity [µS/cm]; *e* is the conversion factor evaluated in the laboratory (e = 0.0006 g*cm/L*µS); and, $C_{baseline}(t)$ is the background conductivity of wastewater during the indicator peak passage [µS/cm].

4. Experimental design

The experimental design concerned the following aspects of tracer dosing: (i) selecting a location of tracer injection along the investigated reach to obtain good indicator and reference signals, (ii) selecting a location for measuring conductivity, (iii) choosing tracer dose, and (iv) deciding whether to overlap the reference and indicator signals.

Concerning the first two points above, the selection of manholes for injecting and measuring tracer does not depend only on the investigated section of the sewer, but also on the location of point inflows into the sewer. Thus a set of continuity and tracer mass balance equations should be developed. For the sewer investigated that set was written in order to define the locations of the points of injection and the measurement (see Section 4.2). A critical aspect of this procedure is the flow model, so before solving the continuity and mass balance equations, different Q(t) functions were studied (Section 4.1) and then used to evaluate the exfiltration ratio by eq. (2). The results are then discussed in Section 5. The model of the flow rate serving to evaluate the exfiltration fraction appeared more reliable if chosen on the basis of the structural state of sewers. It was used to solve the set of equations in Section 4.2.

The dosed amount of tracer should be such that the peak-baseline ratio is as high as possible, but salt precipitation is avoided (360.0 g/L at 20°C [4]).

Regarding the fourth point listed above, the analysis of baseline and discharge trends was carried out. The natural conductivity of wastewater and the flow rate were recorded for three days in order to identify the period of the day when they were as steady as possible. During this period it is possible to avoid the overlapping of peaks, because the baseline can be modelled satisfactorily with a linear function and the unknown flow during the tracer passage can be obtained from the reference peaks measured just after and before the indicator peak. An advantage of using the QUEST method without overlapping is that the peak fitting does not have to be used [2], but some preliminary tests need to be carried out to determine the shape of the peak at the end of each investigated reach. The knowledge of the peak shape is needed to evaluate the time interval between two subsequent tracer injections.

4.1 FLOW RATE MODELS

The models of the flow rate during the indicator passage can be developed using the data from both reference peaks and the in-line probe for discharge measurement. The following models can be used for this purpose:

1. the mean value of flow rate measured by reference peaks (see Fig. 4 - a);
2. linear regression between the mean value of flow calculated from the two reference peaks before the indicator peak and the two peaks after it (see Fig. 4 - b). The equation for the calculation of flow rate from the reference peak is:

$$Q_{ref} = \frac{M_{ref}}{\int\limits_{span\_ref.} (C(t) - C_{baseline}(t))dt} \tag{3}$$

3. discharge measured by means of the submerged probe (see Fig. 4 - c);
4. discharge measured by the probe, of which reading is corrected by factor $\overline{K}$ (see Fig. 4 - d):

$$K_i = \frac{Q_{ref\_i}}{\overline{Q_{meas}\left(\Delta t_{Qref\_i}\right)}} \tag{4}$$

i = 1 : number of reference peaks

$$\overline{K} = \frac{\sum_{i=1}^{n\_ref.} K_i}{n\_ref} \tag{5}$$

Each model above (called the Q(t) function) was used in eq. (2) to estimate the exfiltration fraction. The first two models should be adopted when it is impossible to use on-line probes in measurements. In particular, the first model is well suited when flow is rather steady, and the second one is well applicable when the flow rate greatly varies. The third model is recommended for use when the discharge strongly varies and it is possible to install flow measuring devices. The fourth is applicable when it is possible to install a flow meter, but its accurate calibration is not possible.

4.2 EQUATION SYSTEM

The system of equations consists of three continuity equations, three mass balance equations and three exfiltration fraction equations. For the sewer investigated (Fig. 5), the equations can be written as:

$$Q_1 = Q_2 + Q_{12} = Q_2 + exf_{12} * Q_1 \tag{6}$$

$$Q_3 = Q_4 + Q_{34} = Q_4 + exf_{34} * Q_3 \tag{7}$$

$$Q_3 = Q_2 + Q_5 - Q_{23} = Q_2 + Q_5 - exf_{23} * (Q_3 + Q_5) \tag{8}$$

$$M_{ij} = M_j + exf_{ij} M_i \qquad \text{i = 1:3; j = 2:4} \tag{9}$$

$$exf_{ij} = \frac{M_{ij}}{M_i} \qquad \text{i = 1:3; j = 2:4} \tag{10}$$

where: Q_1 = inflow rate [L/s]; Q_2 = flow rate measured at point 2 [L/s]; Q_3 = flow rate measured at point 3 [L/s]; Q_5 = flow rate from a lateral inflow [L/s]; Q_4 = flow rate measured at point 4 [L/s]; M_1 = tracer mass injected [g]; $M_{2,3,4}$ = tracer masses measured [g]; Q_{12} = exfiltration rate from the first reach; Q_{23} = exfiltration rate from the node; Q_{34} = exfiltration rate from the second reach; exf_{12} = exfiltration fraction from the first reach; exf_{23} = exfiltration fraction from the joint; and, exf_{34} = exfiltration fraction from the second reach.

The model is based on an assumption that the tracer is completely mixed 100 m downstream from the injection manhole, and further downstream from this point, the loss of tracer equals the loss of wastewater.

In the above set of equations, there are five degrees of freedom, so five values need to be determined by experimental measurements. The experimental procedure was set up (see Fig. 5) as follows: (1) measuring at point 2 to determine M_2 and Q_2 by dosing the tracer at points I_1 and R_2; (2) measuring at point 3 in order to determine M_3 and Q_3 by dosing the tracer at points I_1 and R_2; and, (3) measuring at point 4 in order to determine M_4 by dosing the tracer at point I_1.

5. Results

The results presented here were produced in three experimental surveys carried out for the application of the QUEST method [2]. The dosed tracer masses were chosen such as to produce peak concentrations 2-3 times higher than the average value of the baseline, for the concentration of tracer solution of 130 g/L, well below the limiting concentration of 360 g/L at 20°C [4]. In Table 1 the results of the three experiments evaluated by using different flow models are shown. Because the exfiltration fractions calculated from the mean values of Q for reference peaks are more reliable for sewers with a good structural state, the system of equations (Section 4.2) was solved using this model of the Q(t)-function. The exfiltration fractions for the sections upstream and downstream of the node are shown in Table 2.

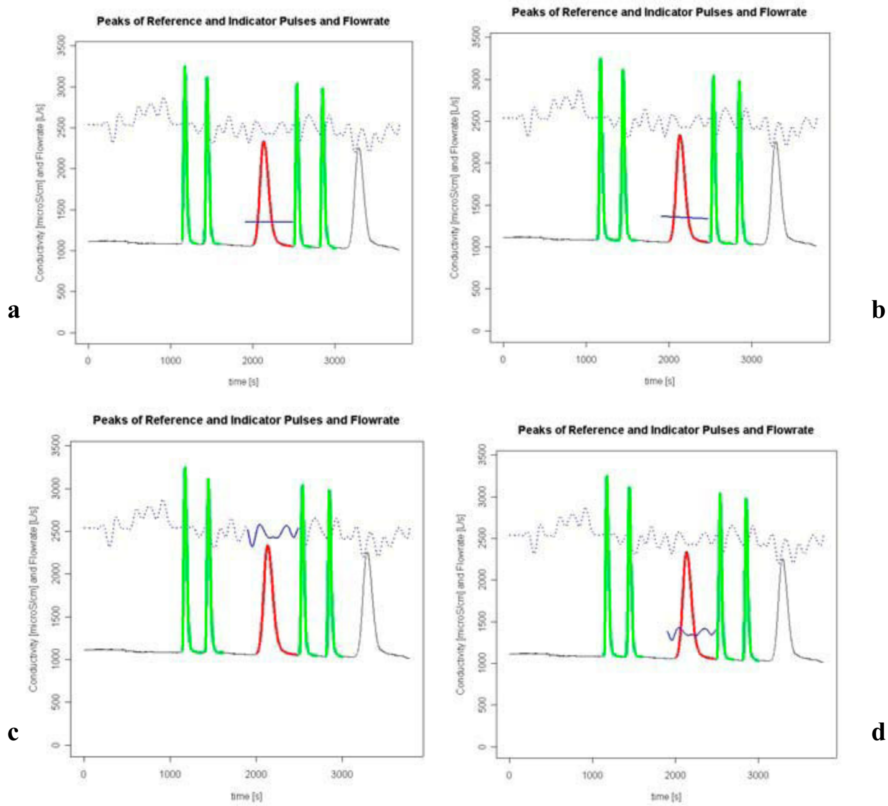

Fig. 4. Conductivity [µS/cm] and flow rate [L/s *100] vs. time [s].

The four panels show different ways to estimate Q(t) during the passage of the indicator peak: (a) the flow rate equals the average value of the flow rate measured by means of reference pulses; (b) the flow rate is determined by a linear regression of average values measured by means of reference pulses; (c) the flow rate is measured by the probe; and, (d) the values of the discharge measured during the i-reference peak passage are scaled by means of average value of factor k (eqs. (4) and (5)).

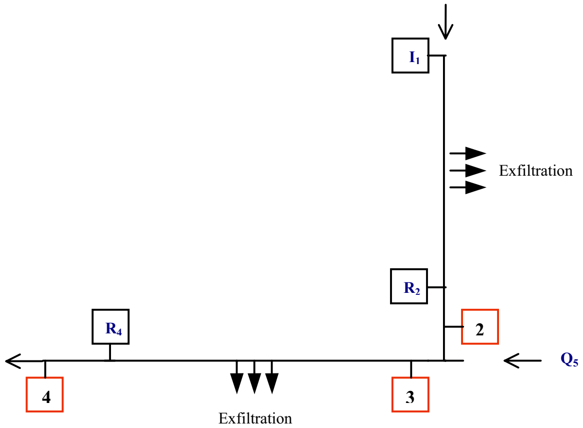

Fig. 5. Sketch of the investigated sewer network with the injection and measuring manholes. I_1 = tracer dosing for indicator signal; R_2 = tracer dosing for reference signal; 2,3 and 4 = measuring points, Q_5 = lateral inflow.

Table 1. Exfiltration fraction [%] evaluated for the entire tested system

	Q mean	Q Linear	Q measured	Q modified by mean K-factor
Experiment 100703	7.08	6.17	-116.150	12.25
Experiment 240703	0.2	0.53	-81.220	5.83
Experiment 300703	3.9	8.62	-66.15	5.89
Mean	3.73	5.11	-87.84	7.99
st.dev.	2.81	3.39	20.94	3.01

In Table 2, the results obtained for models A and B, respectively, are shown.

Table 2 - Exfiltration fraction [%] evaluated for the two tested reaches

	10th July 2003	24th July 2003	30th July 2003	Mean	st.dev.
First Reach	-0.123	-0.235	-0.211	-0.190	0.059
Second Reach	2.959	2.157	2.137	2.418	0.469

6. Discussion

In this section, the data presented in Tables 1 and 2, and the principal sources of uncertainties are discussed.

The results shown in Table 1 correspond to the exfiltration fraction for the entire investigated sewer, but calculated by using various Q(t)-functions. The fraction varies greatly; the large negative values are caused by overestimation of discharge due to systematic errors in the measurements. The results vary during the individual surveys, because eq. (2) is very sensitive to the choice of the Q(t)-function and the function output is further affected by several factors. Some of these factors do affect the leakage (e.g., the wastewater flow level, soil saturation, etc.), while others reflect such sources of uncertainty as:

1. Conductivity peak shape
2. Peak distance
3. Natural conductivity of wastewater (baseline)
4. Q(t)-function during the passage of the indicator peak
5. Solids in sewage (e.g., toilet paper, plastics, etc.) and flow turbulence, and
6. Tracer mixing.

So a special care has to be taken to reduce the effect of these factors.

6.1 PEAK SHAPE AND TIMING

In order to reduce the uncertainty in the determination of the start and end points of the peak records, some preliminary testing has to be carried out in the investigated reach before proceeding with the experiment. Furthermore, from these tests, the duration of the peak has to be determined in order to avoid the peak overlap and to decide the timing for the tracer solution injection during the experiment. The experiment duration has to be as short as possible, because the shorter the experiment, the smaller the variability of natural conductivity and flow during a quasi-steady period of the day.

6.2 BACKGROUND CONDUCTIVITY

The errors due to the variation of the natural sewage conductivity can be reduced if the experiment is carried out when the conductivity is steady. So the experiment was carried out when the maximum variability was not more than 100 µS/cm.

In eq.(2) the function $C_{baseline}(t)$ is unknown during the peak duration and it has to be modelled. The model used was a first-order polynomial using 200 data points equally distributed before and after the peak (see Table 3). Since the number of regression points influences the distance between the peaks, it was desirable not to consider more than 200 data points in order to reduce the duration of the experiment as much as possible.

6.3 Q(t)-FUNCTION DURING THE PASSAGE OF THE INDICATOR PEAK

The discharge variability during the experiment and the availability of a suitable site for the installation and calibration of the flow meter determine, which of the Q(t)-functions to use in eq.(2). Eq.(2) is very sensitive to the choice of the Q(t)-function (see Table 1). If no other discharge measurements are available, the flow rate can be measured by means of the reference signals and the overlapping QUEST method is recommended. Otherwise, to apply the non-overlapping QUEST method, measurements over three to four days need to be carried out to find the time of the day when the flow is steady. The more variable the flow, the less reliable are the flow values calculated from reference peaks just before and after the indicator peak.

Table 3. Actual* and estimated** areas under the indicator peak overlapped at natural conductivity background and the error [%] between the actual area and the estimated one.

	Area under the peak [µS*s/cm]	**Relative Error [%]**
Real Area	190936.000	0
Number of regression points		
200	192243.613	0.680
100	192915.689	1.026
50	193205.760	1.174

* Calculated by subtracting the background conductivity from the peak.
** Calculated by subtracting the modelled conductivity from the peak.

The manhole used as the measuring station allowed the installation of the flow meter, but its calibration was not accurate. In the experiments on July 30 and 24, the maximum discharge variability was 5 L/s, while during the experiment on July 10, it was 12 L/s. Thus, for the experiments carried out on the 30th and 24th July, the reliable exfiltration fractions are those which were calculated by considering the flow rate linearly averaged for the four reference peaks (Section 4.1). For the experiments carried out on July 10, the reliable values are obtained by scaling Q(t)-function by a K-factor (Section 4.1).

6.4 SOLIDS IN THE FLOW AND FLOW TURBULENCE

Suspended solids and air bubbles may disrupt conductivity measurements. Therefore, the conductivity probe was protected by metal netting wrapped around the probe and the measuring cross-section in the sewer was located far away from any sewer junctions or invert drops.

6.5 TRACER MIXING

To obtain reliable exfiltration values by the QUEST method [2], the loss of the tracer mass has to equal the loss of wastewater, and the length of the investigated sewer has to be such as to ensure complete mixing. To confirm that this was the case in this study, two probes were installed in the sewer and the coefficient of variation of their readings was 0.98 %, which is a minimum value suggested by Rutherford [3].

The magnitudes of exfiltration obtained from model A (Table 2) were expected, because (a) the tested sewer pipes are relatively new, (b) there is no traffic load on the ground overlying the sewer, (c) the sewer is laid in tuff, and (d) the ratio of the wastewater depth to the pipe height was low (Table 1). The negative values of the exfiltration fraction in the upstream reach are caused by the errors occurring during the measurements and data analysis, rather than by additional tracer sources, because there are no house connections along the investigated pipe reach that might discharge salt.

7. Conclusions

Preliminary results were presented for estimating exfiltration from an urban sewer system by a novel method QUEST, which was developed by EAWAG in the European APUSS project. The application of this method to a structurally sound sewer in Rome proved that the method allows the assessment of exfiltration in an expedient and economic way. These results have provided reliable exfiltration rates on the basis of a sewer structural state. However, the QUEST method can not replace the use of common conventional techniques (e.g., CCTV) for finding exact locations of pipe defects. The paper highlighted the importance of preliminary testing (peak shape study, conductivity and flow rate measurements) and sewer system characterisation, in order to reduce the uncertainty in the results obtained from the proposed models.

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URBAN WETLANDS IN BELARUS: STATE, THREATS AND PERSPECTIVES

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1.Introduction

Wetlands are widely spread in Belarus occupying about 14.2% of the total area of the country (these are data only for wetlands with peat – peatlands). They are also typical elements of landscapes of cities, especially of newly built-up areas. According to somewhat dated land use statistics, there are wetlands practically in every city; they occupy from 0.1 to 5.6% of city territories. Now they show significant transformation resulting from urban development. Preserved wetlands are used spontaneously, because their status is not defined, and no special protection measures are applied to them. These wetlands suffer from intensive anthropogenic influences such as recreation, slides of wetland banks, dumping garbage either near the wetlands or partially in the wetlands, fires, dumping polluted substances, and etc. Nevertheless, many existing wetlands have features that are unique for urbanised territory, and which would be highly important to be preserved in their natural state.

One of the contemporary tendencies in urban development is maximum preservation of environmental diversity, including preservation of natural areas or parks reflecting well the regional characteristics [1-4]. During the last two decades a lot of supporters of peatland protection and their preservation in the natural state emerged, and greater attention is being paid to the biospheric functions of peatlands [5]. One of the most important functions in the urban environment is considered to be the hydrological or water protection function (water storage and redistribution of outflow, enhancement of surface and groundwater quality). Disappearance of small streams in towns is caused to a considerable extent by drainage and elimination of wetlands. Wetlands reduce discharge of suspended solids, nitrogen, phosphates and others pollutants from stormwater into receiving waters.

2. Study objectives and methods

The main objectives of our research were to evaluate the condition of peatlands in the urban areas of Belarus and to clarify their future functions. Specific tasks included the following:

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- peatlands inventory and identification of their area;
- peatlands mapping within the current and future urban borders;
- indicative factors and reasons for peatlands transformation;
- evaluation of the present condition of peatlands and their classification;
- identifying and evaluating the role and functions of peatlands in city landscapes; and
- development of recommendations for peatlands preservation and use.

In the process of investigations, we used the following sources of data: official statistics data (Land Survey of Cities; Peat Fund); agency materials (geological and soil district maps, soil maps of forests, samples and analyses of peat deposits); and, topographic maps of various years of publication.

For more precise definition of the status, and also for evaluation of the current state of peatlands, field investigations were carried out. The objects of our research were wetlands located in several cities of Belarus (Minsk, Gomel, Polotsk, and Svetlogorsk). The total number of the wetlands we have researched was about 20 and all were of various origin and current state.

The research tasks included identification of the most probable sources of influence, the description of the state of vegetation, and the sampling of surface waters and groundwater, soils and vegetation. Hydrochemical indices, botanical composition and chemical properties of peat, and the structure of vegetation were also examined. The chemical composition of urban surface runoff from various land use discharged into the wetlands was studied. Characteristics of natural (unaffected) peatlands of Belarus were used as a reference for assessing of the degree of urban peatlands transformation.

3. Results and Discussion

3.1. DISTRIBUTION AND CURRENT STATE OF PEATLANDS

The most thoroughly studied peatlands were located in Minsk. As a result of the research we drew the maps of distribution of peatlands for years 1994 and 1998. According to our estimation, initially there were about 20 peatlands within the city limits with an area of about 1,000 hectares, which represented about 5% of the city area. We noted 10 peatlands within the city limits, which function in more or less natural conditions. Besides these, we have identified some peatlands, which are being restored after peat extraction and harvesting have been finished. The total area of natural and restored peatlands, located in separate plots, is not more than 110 ha.

Transformation of different natural complexes, including peatlands, into urban landscapes was connected with an intensive development of Minsk and construction activities after the World War II (after 1945). Thus, fen bogs Iokhimovo, Sasnyagi and a high bog Ozerische-1 were completely filled and built-up, fen bogs Akopje and Vesninka were completely flooded, fen bog Tsnyanskoje was transformed into arable land, and a transition bog Komarovskoje was built-up and partially turned into a park. It was noted that no peatlands have been preserved in the central part of the city. However, some peatlands have been preserved in the suburbs with new residential areas (there has not been enough time for their development), and also in the floodplain of the Svisloch River, because the peatlands of this type are hard to transform. Similar findings were made in other Belarusian cities.

The remaining peatlands are rather diverse. First of all their characteristics are derived from the specific conditions of their formation, be it in floodplains, hollows, lake basins or narrows of water bodies. Secondly, the characteristics are related to the type of nutrition, including eutrophic (fens, mires), mesotrophic (transition bogs) and olygotrophic (high bogs), with various depths of peat and types of vegetation [6-8]. Some peatlands are integrated with small lakes. In the overall rating, typical peatlands are usually small (Table 1).

Table 1. Overall performance of preserved peatlands in Minsk

Type of peatland, area [ha]	Vegetation	Peat capacity [m]	Location, usage	Types of impact
High bog Drazhnya, 1	Pinus-Eriophorum-Sphagnetea	2.7	500 m from a tractor plant, bordering with a park zone and allotment gardens; partially built-up	Atmospheric precipitation, littering of the banks, discharges from houses
High bog Drozdy, 8	Pinus-Vacciniun	3	In the green zone, forest park, drained	Atmospheric precipitation, recreation
High bog Mokhovoe, 10	Pinus-Ledum-Eriophorum-Sphagnetea	5	Green zone, 1.5 km from a city ring highway	Atmospheric precipitation, recreation
High bog Bogdanovskoe, 15	Betula-Vacciniun	6	500 m from a ring highway, partially crossed over	Fires
Transition bog Sukharevo, 1.2	Carex	0.4	Surrounded by agricultural land, a new development is nearby	Burning of grass in spring
Transition bog Kuntsevschina, 1.5	Alnus	1.2	Eastern part was destroyed for road construction, drained	Road building, littering of the banks
Transition bog Tsna, 1.5	Salix-Pragmites-Equisetum	1	Near a private development district, contains a dump in the eastern part	Discharges of wastes
Floodplain Loshitsa, 30	Equisetum-Carex	1	Floodplain of the Svisloch River	Littering of the banks
Floodplain Sheipichi, 14	Alnus	0.7	Floodplain of the Svisloch River	Stormwater runoff, littering
Floodplain Dvorische	Equisetum-Carex	1	Floodplain of the Svisloch River	Fishing, littering
Former transition bog Masjukovskoe, 50	Mixed	Up to 2	Near an open market; restored after construction of a crossing; bird refuge	Discharges from an open market Zhdanovichi
Former high bog Ozeristsche, 8	Mixed	0.4-1	Restored after construction of a crossing	Road runoff, littering

The distribution of wetlands (peatlands) in the Minsk Metropolitan Area is shown in Figure 1.

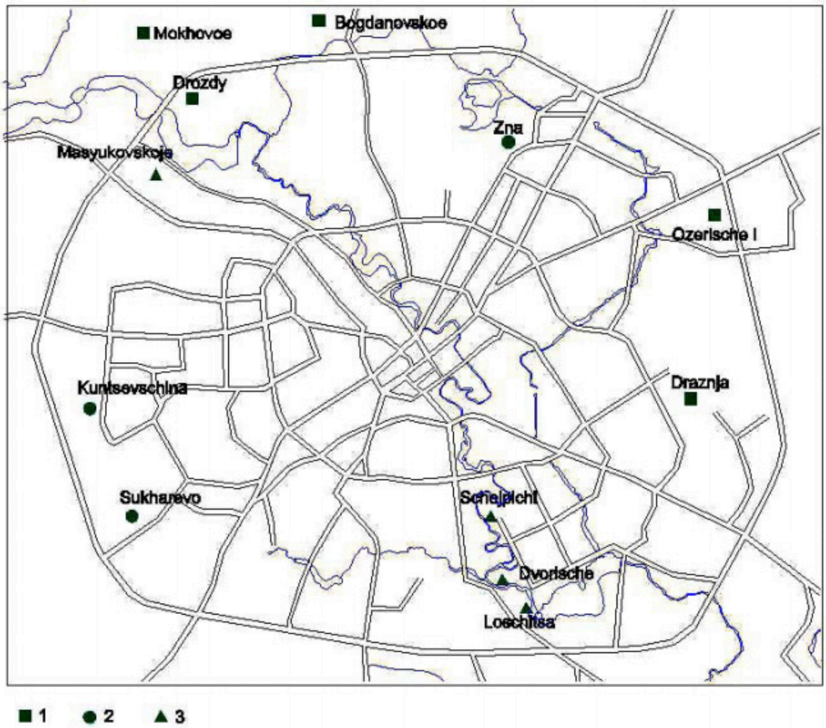

Figure 1. The preserved wetlands in the Minsk Metropolitan Area: 1- olygotrophic, 2 – mesoeutrophic, 3 - eutrophic

3.2. SOURCES OF POLLUTION

The experience from an ecological-geochemical study of the cities of Belarus has shown that the general state of soils and waters depends mostly on the type and intensity of land use, the level of land maintenance and the presence of local sources of pollution [9]. Thus most polluted soils and waters were found in industrial zones and agricultural areas, where they are polluted by sewage and stormwater, solid wastes of industrial, construction and domestic origin, use of ash as a fertiliser, open burning of domestic and agricultural wastes (this is a very common practice in small and medium size towns).

The above sources can contribute, to some extent, to heavy metals, polycyclic aromatic hydrocarbons and other pollutants emissions. Such sources as ashes are affected both by the diversity of substrates exposed to fire and by the specificity of the burning conditions. According to [10], the content of zinc in some ash samples was as high as 1-30 g/kg, which exceeded 10-300 times the background levels in soils.

It was established that residual ash in fire-bones contains large amounts of soluble components (up to 42 g/kg of ash), including sulphate (2.0-21.5), chloride 0.04-20.5 g/kg ash), etc.

3.3. TRANSFORMATION

The state of wetlands in cities and suburbs differs significantly. It depends on peatland genesis and on the type of nutrition as well as on peatland location and sources of pollution. For example, the groundwater of high bogs (Bogdanovskoe, Drozdy, and Mokhovoe) is acidic, with low mineral content (Table 2). Vegetation of the remaining bogs is characterised by typical oligotrophic associations (*Pinus-Ledum-Eriophorum-Sphagnetea*, *Ooxycocco-Sphagnetea*, *Pinus-Eriophorum-Sphagnetea*, *and Pinus-Vaccinium*).

Table 2. Content of main ions in subsoil and surface water in urban wetlands, mg/L

Peatlands	Water type	pH	HCO_3^-	Cl^-	SO_4^{2-}	NO_3^-	NO_2^-	Ca^{2+}	Mg^{2+}	Na^+	K^+	NH_4^+	Ion sum
Bogdanovkoe	Subsoil (6)	4.8	38.1	9.1	3.6	0.8	0.0	8.0	3.9	1.4	1.3	9.1	75.4
	Surface (11)	6.2	72.4	11.4	36.3	3.2	0.0	24.3	6.5	2.7	2.1	4.5	163.1
Draznja	Subsoil (5)	4.2	65.9	19.0	12.1	1.0	0.0	15.8	6.2	8.0	4.6	5.3	137.7
	Surface (8)	7.0	144.5	30.9	9.5	1.7	0.0	28.3	9.9	17.8	18.5	1.2	262.3
Drozdy	Subsoil (5)	3.7	61.0	20.6	8.3	1.7	0.0	9.1	4.4	1.6	0.9	8.3	114.3
Mokhovoe	Subsoil (8)	4.0	23.0	15.2	23.3	1.9	0.0	6.2	3.5	2.1	1.8	7.9	82.7
	Surface (2)	5.1	33.6	9.8	8.0	1.1	0.0	6.8	4.2	0.8	2.1	3.7	69.9
Kuntsevchina	Subsoil (7)	5.2	67.9	31.3	48.9	2.1	0.0	22.2	9.3	13.1	7.8	4.8	207.2
	Surface (5)	7.3	111.7	18.0	28.1	1.0	0.0	31.0	10.3	14.6	6.6	1.8	227.9
Sukharevo	Subsoil (2)	5.8	30.5	17.7	13.7	0.5	0.0	6.1	6.8	1.9	5.4	2.7	85.4
	Surface (8)	6.5	47.6	11.6	14.9	1.7	0.0	5.4	3.8	3.8	18.0	2.1	108.8
Tsna	Subsoil (5)	7.1	239.1	118.2	3.8	0.5	0.0	58.3	16.7	99.4	5.4	1.6	543.0
	Surface (9)	6.6	190.4	104.9	6.7	0.4	0.0	42.0	9.2	69.7	6.3	1.3	432.7
Loschitsa	Subsoil (12)	6.6	294.6	15.8	28.3	0.6	0.0	68.7	21.3	8.1	2.6	0.8	440.7
	Surface (14)	7.3	251.8	21.0	21.2	1.3	0.1	57.6	18.9	9.4	3.4	0.5	384.9
Schejpichi	Subsoil (4)	6.6	297.4	53.2	46.1	1.1	0.1	88.0	23.1	20.5	0.7	1.2	531.3
	Surface (6)	7.3	325.7	50.8	15.2	0.5	0.0	76.9	24.6	24.3	3.7	1.2	523.1
Dvoristche	Subsoil (2)	6.7	305.0	7.8	6.9	0.4	0.0	59.4	18.7	6.7	1.2	2.4	408.4
	Surface (4)	7.1	266.9	11.2	3.5	0.4	0.0	57.1	20.2	5.1	0.4	0.7	365.4
Masjukovskoe	Subsoil (9)	6.1	235.1	61.9	146.9	2.1	0.0	104.4	24.6	16.9	0.9	16.6	609.4
	Surface (22)	7.5	226.2	67.1	50.2	0.6	0.1	71.2	24.3	22.2	1.6	0.8	442.7
Volovo, outskirts, built-up	Subsoil (2)	8.0	671	95.1	245.5	18.8	0.0	186.2	41.7	123.8	33.0	5.6	1420.8
	Surface (5)	7.8	486.8	81.5	59.7	1.0	0.0	117.1	30.0	57.6	26.2	1.7	862.8
Volovo, centre, undisturbed	Subsoil (2)	7.3	286.7	65.3	11.5	0.8	0.0	59.8	20.2	50.5	14.5	2.3	511.6
	Surface (37)	8.2	168.8	61.7	37.0	0.4	0.0	34.1	18.8	40.7	19.2	0.8	397.5

Also the hydrochemical characteristics of floodplain peatlands (Loschitsa, Schejpichi, and Dvoristche) are close to the background values. But what makes floodplain peatlands really beautiful is their rich flora. About 50 mire plant species were observed in the peatland Loschitsa, including rare species that are registered in the Red Book of Belarus *(Dactiloriza baltica Aver., Listera ovata R.Br.).*

The most significant changes were observed in those peatlands, which were located near industrial and residential areas and were partially built-up or used for waste dumping (high bog Drazhnja, transitional bogs Tsna, Kuntsevshina, Volovo). In these cases, contaminant loads originated from large volumes of discharges from houses and from the littering of the banks. Peatland waters contained elevated concentrations of almost all constituents, but especially of chlorides, potassium, ammonium nitrogen and phosphate [11].

Ash content in peat reaches up to 15-20%; the content of saline components in subsoil and surface waters has also increased. As a result, ruderal and synantrophic plant species appear. For instance, in the high bog Drazhnia such species as *Salix aurita, Salix cinerea, Salix pentandra, Betula pubescens, Galium uliginosum, Politrichum commune* have appeared. Moreover, on the edges of the bog, *Tuncus efbusus, Calamagrostis canescers, Ayrostis stolonifera, Equisetum fluviatile,* etc. were found. At the same time, typical oligotrophic plants (cranberry, great bilberry, sphagnum mosses, etc.) disappear.

When considering the pollution and transformation of urban wetlands, it is necessary to note that in some cases the pollution of wetlands prevents the pollution of rivers, lakes and groundwater. The ability of peat to act as a pollution barrier was illustrated by the Volovo peatland, where differences (2-3 times) in chemical composition of water between the outlaying areas and the centre were established.

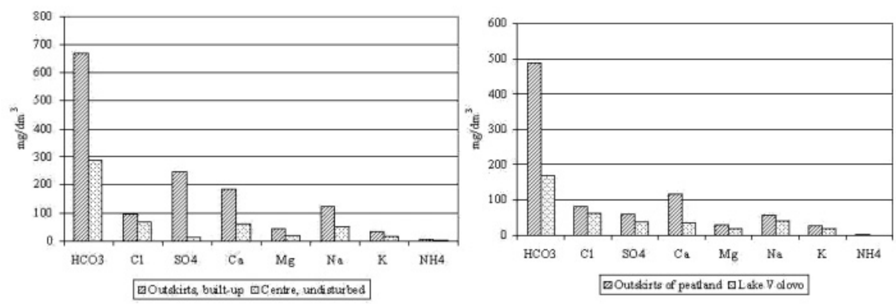

Figure 2. Chemical composition of subsoil (on the left) and surface water (on the right) of Volovo peatland (The Town of Polotsk)

Investigation of the stormwater runoff from an open market to the Masjukovskoe wetland has shown that wetlands intercept runoff and treat it by removing suspended solids (Figure 3). In other words these wetlands are a natural barrier to transport of suspended solids into the main city river, the Svisloch River.

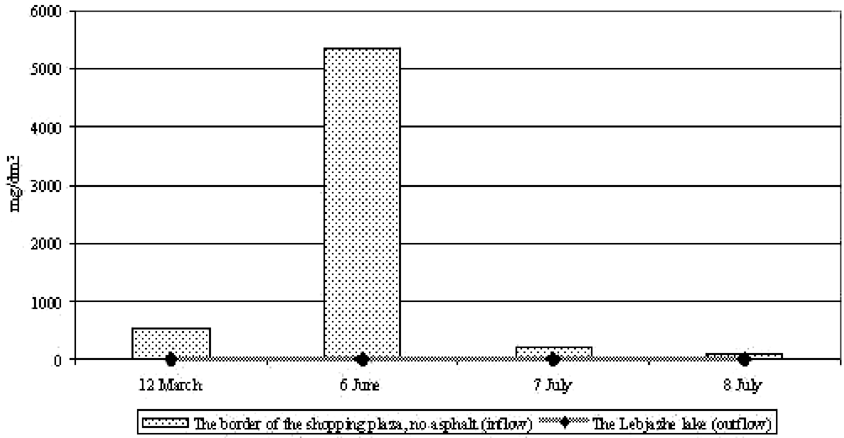

Figure 3. Concentration of suspended solids in stormwater runoff from the open market Zhdanovichy (inflow to Masjukovskoe) and in the Lebjazhe lake (outflow to the Svisloch River)

We have also studied functions of peat in sorption of heavy metals in the Volovo peatland, where peat was sampled along the highly polluted perimeter and in the central part. The distance between sampling points was about 100 m. The reduction in zinc content in surface peat between the outlaying areas and the centre in the Volovo peatland is shown in Figure 4.

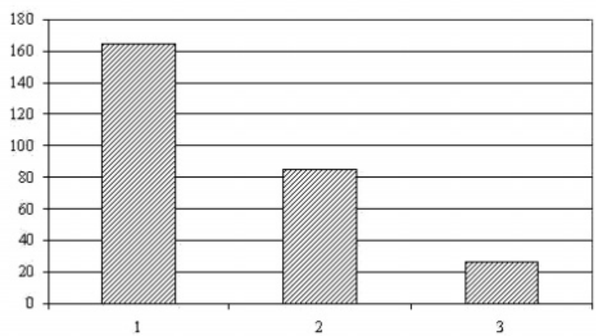

Figure 4. Decreasing zinc content (mg/kg) in peat (0-10 cm) progressing from the perimeter to the centre of the Volovo peatland: 1 – built-up outskirt; 2 – 100 m from boundary inward; 3 – centre of the peatland

It was also observed that the anthropogenic impact was apparent only in surface peat layers (Figure 5).

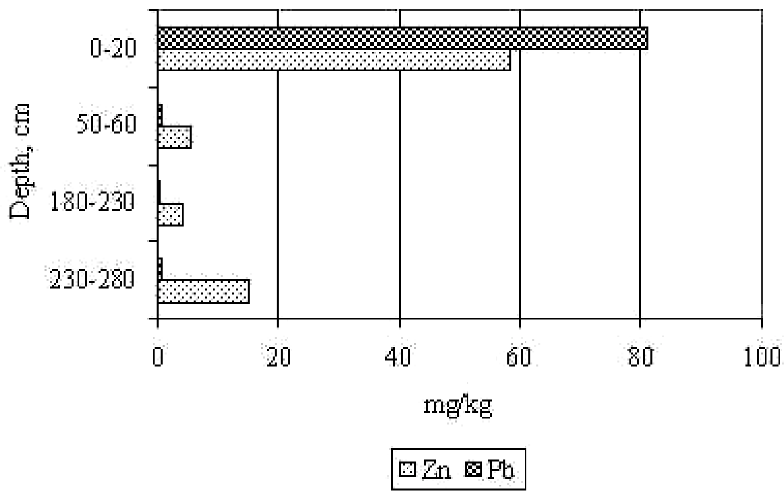

Figure 5. Content of zinc and lead in peat (Volovo peatland, Town of Polotsk)

3.4. CONFLICTS

In general there is a paradoxical situation regarding urban wetlands. On one hand, the attitude of the city government and city planning department toward them continues to be negative, and wetlands are still disappearing. On the other hand properly preserved wetlands represent a piece of actual “wilderness” in cities, because in some cases, high water level, difficult to cross brushwood and floating mats of vegetation make these wetlands impassable for citizens.

It has been established that quite often problems related to wetlands occur only because urban wetlands are viewed as "useless" natural objects.

It goes without saying that, in a conventional understanding, a bog and a city are two totally incompatible concepts. Nevertheless, we look at peatlands in urban areas as small remnants of nature, which do not cause any great discomfort to urban dwellers. Our research has indicated that the majority of peatlands in cities are so small and so harmoniously placed in the surrounding landscape (in combination with ponds or forests) that only real lovers of nature are aware of their existence. On the other hand, real negative influences, or discomfort to the population, are created when houses are built directly on the peatland. For example, in some cities (Borisov, Berezovka, Polotsk) peatlands could not be modified to provide comfortable living and many built-up areas were flooded during snowmelt or heavy rainfall.

Sometimes the negative attitude towards peatlands can be caused by their unattractive appearance, which in turn was caused by construction works. Peatlands and waterlogged areas are of little use for construction and that is why they are used for development only as a last resort. Moreover, in some cases they appear under construction without any special development status or specific owner. After the construction works around a bog are completed, builders usually leave behind ugly made road depressions, mounds of dirt that are soon covered with tall weeds, holes in the ground and other inconveniences.

Among other factors that influence the attitude towards peatlands (and towards the nature as a whole) we can name low living standards of population and correspondingly low level of culture, and predomination of state property.

One cannot view peatlands as an alternative to some other piece of city infrastructure, it (as any other object of nature) should not cause discomfort or be viewed as an obstacle. Overall, local population is rather indifferent when it comes to the improvement of the environment, especially if it is beyond their personal interests.

3.5. WISE USE OF WETLANDS IN URBAN AREAS (CLOSING OBSERVATIONS)

With progressing urbanisation the necessity for developing new areas is also increasing. As a consequence urban wetlands can be easily destroyed (drained, filled up with soil or wastes). That way, the last remnants of nature in cities disappear. In order to prevent this situation, we suggest a number of option for integration of preserved wetlands into the urban landscape.

For example, the typical high bog Drazhnya, with unusual (for a city) olygotrophic plant species, and the floating bog Kuntsevschina can be turned into protected conservation areas. Some peatlands can form a recreation area in new residential areas, for example, peatland Sukharevo that is connected with a small reservoir and peatland Tsna that is connected with a meadow with a high variety of herb species. In the Loshitsa peatland which has the largest variety of vegetation associations, the construction of an ecological path (an elevated boardwalk) for school children and students of biology is recommended.

Finally, some small urban wetlands (with lakes, submerged narrows) could be used for stormwater treatment by removal of suspended solids, nitrogen, phosphates and others pollutants (Scheipichi in Minsk, some wetlands in Polotsk). The existing wetlands with attractive ruches and reeds could be further aesthetically enhanced by addition of blooming and decorative plant species.

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RESTORATION OF URBAN WATERS IN ST. PETERSBURG: SOCIAL, TECHNICAL AND FINANCIAL ASPECTS

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1. Introduction

St. Petersburg is located at the mouth of the Neva River and across the islands of its delta on the coast of the Finnish Gulf of the Baltic Sea. The territory of the city is 606 square kilometres. The population of St. Petersburg is 4 780 000 inhabitants. St. Petersburg, due to its own unique architecture and beautiful suburbs with the world famous palace and parks, is recognised as a cultural capital of Russia. In 2003 St. Petersburg has marked three hundred years since its foundation.

The city is located on 42 islands. The main water bodies in the city include the Neva estuary, the Neva River with its tributaries and branches, other rivers and channels, natural and artificial lakes and reservoirs, and marshes. There are 64 streams, 48 channels and 34 brooks in St. Petersburg and its suburbs. Within the city, there are more than 130 reservoirs with the water surface area greater than 1 ha. The characteristics of basic streams are given in Table 1.

St. Petersburg is obviously one of the cities of the world with the greatest abundance of water. Almost 20 % of the city territory is covered with water. If the Neva estuary (serving now as a city reservoir) in the Finnish gulf were included in the city area, the share of the water surface would represent 50% of the city area. St. Petersburg is subject to flooding. Catastrophic floods occurred in 1824 and 1924, when the level of water rose above the mean level by 4.2 and 3.7 m, respectively. The highest point of Vasilievsky Island has an elevation 3.80 m, Petrogradsky Island 3.40 m. For protection of the city against floods, a dam was constructed along the western boundary of the Neva mouth. Eventually, a complex of flood protection structures will stretch through the entire Neva estuary from the south to the north, over a length of 25.4 km, including a section of 22.2 km over water. The structure will consist of 11 rockfill/earthen dams, with six openings for flow of water and two for ship passage. The main river is the Neva, with an average discharge of 2,500 m^3/s. The Neva is a source of drinking water supply to St. Petersburg. Municipal wastewater is treated and the treated effluent is discharged into the Neva estuary in the Finnish gulf. The area of the estuary is 329 km^2, with prevailing depths of 3 -5 m.

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Table 1. Rivers and streams in St. Petersburg

River/Stream	Length [km]	Width [m]	Depth [m]	Discharge [m^3/s]
Neva	74	400 – 1300	3 – 24	2,500
Large Nevka	8.5			63.0
Small Nevka	4.9	120 – 300	3.6 – 6.8	240
Small Neva	4.3	200 – 375	2.9 – 7.7	480
Karpovka	3.0	15 – 25	1.6 – 2.9	2.0
Tchernaya	8.1			0.1
Fontanka	6.7	35 – 70	2.6 – 3.5	22.0
Moyka	4.7	20 – 40	2.1 – 3.2	11.0
Ghdanovka	2.2	35 – 65	2.2 – 4.0	14.0
Ighora	76			12.1
Slavyanka	39			1.8
Obvodny channel	8.1	20 – 65	1.8 – 3.8	15.0
Ochta	90	11 – 45	1.0 – 2.7	7.2

Sewer system on two thirds of the city territory is combined and the remaining part is served by a separate system. Currently, long-term development, construction and reconstruction of the sewer system are nearing completion. About 70% of wastewater from the city is subject to full biological treatment. By 2005, up to 80% of wastewater from the combined sewer area will be treated, and the treatment of all wastewater is planned by 2015. Reaching this ambitious goal will require large investments, material resources and hard work. It will be necessary to construct major intercepting tunnel collectors, connect to them sewers conveying untreated wastewater, reconstruct sewer network, and build a new facility for sewage sludge incineration.

The condition of urban waters is an objective indicator of the degree of pollution of the environment. The natural processes of water quality and functioning of aquatic ecosystems in urban areas are generally disturbed because of a high level of anthropogenic pollution loading and low self-purification of urban streams and reservoirs. The poor quality of water negatively influences aquatic life and the health of urban dwellers.

Effective operation of urban aeration facilities and construction of local wastewater treatment plants (WWTPs) by the city during the 1980s and some reduction in the share of industrial wastewater in total wastewater flow have ensured some recovery of the quality of the Neva River and the Neva estuary during the 1990s. Ecological conditions in these water bodies have stabilised.

2. History

In the 19th century, St. Petersburg had no sewers. The wastewater was disposed in special waste pits with walls made of timber or stone. House owners were responsible for cleaning pits by removing their content and disposing it either in gardens or at waste disposal sites, which were frequently located in the city area. In summer, some waste was disposed into waterways.

In 1818 the construction of a closed sewer for collection and transport of wastewater begun, but because of the lack of funds, it was not completed. With growing city population and the increasing charges for waste transport, the owners of houses began, at their own discretion, to connect waste pits to street drainage pipes and dump waste in the rivers and channels of the city. The intensive pollution of water streams began. At the same time, there were many drainage channels and waterways in the city. However, because of low slopes and flow rates of many channels, waste discharges led to fast stream eutrophication and greater worsening of ecological conditions. Consequently, many channels were filled up.

Widespread pollution of the rivers and channels, and simultaneous contamination of street drainage pipes have compelled the government to issue a law in 1845, which forbade to connect yard waste pits to street drainage pipes. However, this law was not firmly enforced and illicit connections of waste pits to street pipes continued.

Towards the end of the 19th century, fast development of a water supply network led to construction of multi-storey houses and widespread connection of yard waste pits to street networks and an even greater pollution of water streams. This type of waste disposal, which became typical for St. Petersburg, completely contradicted elementary and technical requirements on sanitation.

The number of wastewater outfalls to the rivers and channels has increased so much that they were actually transformed into open sewage collectors, crossing the city territory in all directions. From the beginning of the 20th century, the continuing deterioration of sanitary conditions of the water system and the loss of ecological balance manifested themselves in progressing anthropogenic eutrophication of the Ladoga Lake, Neva estuary and the Finnish gulf. The worst situation in inland reservoirs and streams in city and in the Neva mouth was observed from the late 1970s to the mid 1980s.

The construction of the sewer system in the City of Leningrad (formerly St. Petersburg) begun in 1925. This work was interrupted by the Second World War and started again in 1944. For the quickest improvement of the sanitary conditions in Leningrad, the project started by building main sewer collectors along the embankments of streams to intercept the existing wastewater outfalls from sewers to streams and offering thereby a method for liquidating the pollution of urban waters by wastewater. The first part of the sewer system serving the central part of the city entered into operation in 1958. Systematic construction of WWTPs in Leningrad and its suburbs started only in the 1960s. Between 1960 and 1985, a large task of connecting the outfalls discharging untreated wastewater to intercepting collectors was carried out, involving more than 1000 outfalls. However, the terms of construction were continuously broken, because of low technical capacity of building organisations, shortage of funding, complex conditions of construction, and difficulties with building in the vicinity of historical buildings. During the 1980s, the construction of a dam for protection of the city against floods in the Neva estuary begun. As a result, the flow regime of the Neva estuary has changed, its capacity has decreased, and the dilution of the treated wastewater in the Neva estuary has worsened. It has complicated the problem of improving urban waters of St. Petersburg (Leningrad). The discontinuation of dumping of the untreated wastewater into waters of the city is one of the basic ecological tasks, which after implementation, will allow improving the quality of St. Petersburg's waters.

During the 1990s, steady improvement of sanitary conditions and stabilisation of ecological conditions of the Neva and its estuary was observed, as a result of improved wastewater treatment, sanitation of reservoirs and streams, and industrial output decline. The ecological situation in the Finnish gulf remains of concern, owing to progressing eutrophication.

3. Current conditions

The condition of urban water resources (rivers, channels, lakes, reservoirs, ponds) is an objective indicator of a degree of pollution of urban environment. The natural processes of preservation of water quality and ecosystem functions in waters of the urban landscape, as a rule, are disturbed because of a high level of anthropogenic pollution loading. The basic reason for deterioration of the ecological condition of urban waters is the dumping of industrial and domestic wastewater, and also intensive recreational use of reservoirs and streams.

Currently, three WWTP treat 2.22×10^6 m^3 of wastewater per day, which represents about 70% of the wastewater produced in the city, all of which should be treated. Other wastewater is dumped without treatment into receiving waters of the city through 380 sewers. Besides these inputs, the receiving waters also receive untreated stormwater from city territory served by separate sewers. To maintain the treatment of all wastewaters from St. Petersburg, to discontinue dumping of untreated wastewaters in streams and reservoirs, and to allow the development of new housing and industry, the three steps listed below have to be taken.

1. Firstly, finish construction of the Southwest WWTP with the capacity of 5×10^5 m^3/day.
2. Increase the capacity of the Northern WWTP up to 2×10^6 m^3 of wastewater per day.
3. Construct intercepting collectors and connect to them 380 sewer pipes conveying untreated wastewater.

In 1997 the European Bank for Reconstruction and Development has approved "The long-term program of development of water sector of St. Petersburg", according to which all pipes conveying untreated wastewater should be connected to intercepting collectors by 2015. This task will require large investments (more than $1 billion USD), modern technology and hard work.

In urban rivers and streams, large differences in the hydrological characteristics lead to greatly varying hydro-ecological conditions. Recent research indicates that the best water quality is now found in the Neva River. It can be explained by the low ratio of the influx of untreated wastewater to the river discharge (i.e., high dilution), high velocity in the river and low temperature of the riverine water. At the same time small streams in St. Petersburg are subject to a high degree of anthropogenic pollution. This is caused by the relatively high input of dumped wastewater compared to the stream discharge, stream low flow rates and small depths. Besides poor water quality, a large impact on small streams is caused by secondary pollution due to intensive sediment deposition and the resulting pollution of bottom sediments.

The pollution of small streams in St. Petersburg represents a serious ecological problem, because they are used in warm season for boating. The most polluted rivers are the Ochta, Slavyanka, Karpovka and Obvodny channel. In 1990, with the help of the grants and technical support obtained from the governments of the Netherlands and Belgium, the dredging of bottom sediments from streams and reservoirs in the central part of the city started. Currently, these measures are considered to be one of the basic practical ways for improving the condition of urban streams and reservoirs. In 2002, the municipal government of St. Petersburg has adopted the program for bottom sediment dredging from St. Petersburg reservoirs during the period from 2004 to 2008.

Unfortunately, there are not enough investigations being done on urban reservoirs (lakes, reservoirs, ponds). By the end of 2001, not more than 15% of the total number of such water bodies was investigated. For most of them, the missing information includes the basic limnological characteristics (volume, water surface area, and depth), sediment deposition characteristics (volume and capacity of depositions, their physical and chemical structure), and the degree of overgrowth, shoreline condition, sources of pollution, and the quality of water. Only for 18 reservoirs, public health related monitoring is conducted. Considering the incomplete information available for reservoirs, their classification has been attempted for all the 118 reservoirs. The following four groups were recognised:

- Landscape reservoirs unsuitable for economic use - 40,
- Reservoirs somewhat suitable for economic use - 48,
- Reservoirs well suited for recreation in the coastal zone - 18, and
- Reservoirs with potential for multifunctional use, suitable for development of recreational activities (swimming, boating, water sports, coastal zone recreation, etc.) - 12.

Large attention is now given to the completion of construction of St. Petersburg flood protection structures. In 1990, as a result of discontinuation of financing, the work on these structures stopped. At the same time, the discussions about the environmental harm caused by dams were initiated. In particular, concerns about the changes of the hydrological regime of the Neva estuary and detention of untreated wastewater, caused by dams. This issue was frequently used even for political goals, typically in isolation from the discussion of measures for the improvement of ecological conditions of the city rivers and streams. However, the research has shown that if the proposed WWTP is built and the dumping of untreated wastewater is stopped, the complex of flood protection structures will not affect the ecological conditions of the Neva and its estuary.

4. Reconstruction of the historical hydrosystems of St. Petersburg

In St. Petersburg and its suburbs, many hydrosystems were constructed in the 18th and 19th centuries for various purposes. The palace and park groupings were created in St. Petersburg and its suburbs were created, with important roles for these water systems. Canals were constructed in St. in Petersburg for transportation and also for drainage of waterlogged lands. Such structures helped to shape the appearance of a young city. In the 20th century many canals were filled with soil, because they were deemed no longer needed, or because of low flows contributing to the risk of eutrophication. Also because of the lack of funding, it was not possible to maintain water systems of the majority of palace parks.

The interest in reconstruction of these structures grew in the first years of the 21st century in connection with preparation of city's celebration of the three-hundredth anniversary. The restoration of historical water systems considerably improves the aesthetic character of the city and increases its appeal to the inhabitants, tourists, and business investors. Thus the designers and builders had to solve a number of complex technical tasks. It was necessary to restore the selected structures in their historical appearance using historical documents, and in that process, to ensure the durability of designs, prevention of eutrophication of rivers, streams and reservoirs, and water-logging of the territory. Large research was required, as frequently there were no historical descriptions and pictures of objects to be reconstructed. In preparation for the anniversary, the water system of the Konstantinovsky palace in Stralnya, where the President of Russia V.V. Putin met with the heads of states arriving for anniversary celebrations, was reconstructed.

The reconstruction of the canal around the Mikhailovsky castle in the centre of St. Petersburg and restoration of the system of ponds and channels in Pushkin and Peterhof has started. The reconstruction was accompanied by work on the ecological improvement of water systems, which is especially important for recreational and tourist use of such water bodies. The completed works on reconstruction of water systems have considerably changed the face of many historical places in St. Petersburg. It is hoped that work on reconstruction and restoration of other historical water bodies, many of which are in the city and its suburbs, will continue after jubilee celebrations. There are causes for optimism, because during the last 2-3 years, a new financing scheme has been developed to finance similar projects. Consequently, 80% of financing for the reconstruction of the Konstantinovsky palace complex was from other than government sources.

5. Conclusions

In St. Petersburg, the work on restoration of urban waters is continuing. It is a complex program, which requires a great investment, and findings solutions to new technical and social problems. At the same time, it is recognised that this work needs to be based on sound water resources management principles. In this process, there is a great need for the monitoring of water resources and the monitoring results should be made available to the experts and the public.

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THE INTERACTION BETWEEN WATER AND SOCIETY

A new approach to sustainable stormwater management

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1. Introduction

The Rio declaration and the Agenda 21 from the early 1990s introduced the concept of long-term sustainability of our environment. One important ingredient in the new approach is that technical, economic and social aspects of the development are addressed in an integrated manner. There is today a consensus that urban water systems should be approached in an integrated way. Surface water, groundwater, water quality, water quantity and ecology should be looked upon in relation to each other. We also know that integrated approaches can give inspiration to new solutions to water problems. Thus, the introduction of the concept of sustainability has led, in the field of urban water systems among others, to an increased interest in source control and open stormwater drainage within the urban environment.

Although the integrated approach sounds attractive, there are many obstacles to its implementation. In recent years many integrated urban water plans have been developed. Interdisciplinary teams develop plans, formulate optimum packages of measures and create beautiful reports with nice pictures. Local politicians often enthusiastically receive these plans. However, many of the suggested measures are never implemented [1]. As some people say: "Integrated urban water plans die of their own beauty."

Urban water managers have become aware that it is not sufficient to restrict an integrated approach to surface water, groundwater, water quality, water quantity and ecology. They also require measures that are attuned to spatial planning, traffic, maintenance and management of public areas, urban renewal, etc. A town displays a great deal of dynamics, in which a large variety of professionals intervene. Traffic engineers adapt roads and streets, architects make new designs for buildings in the city centre, green belts are changed, programs are initiated to improve social safety, and town planners implement urban renewal plans, in which significant parts of the town are redesigned. It is essential that water management fits into this dynamics. However, even today, that is not always the case.

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2. Two planning approaches

Traditional planning models for urban water systems are to great extent based on technical and economic considerations. Thus, urban water systems are mostly regarded as controlled systems that have to be optimised: we measure, define objectives, compare measurements and objectives, remove bottlenecks and apply techniques to improve the water system. Finally, the system will attain the objectives.

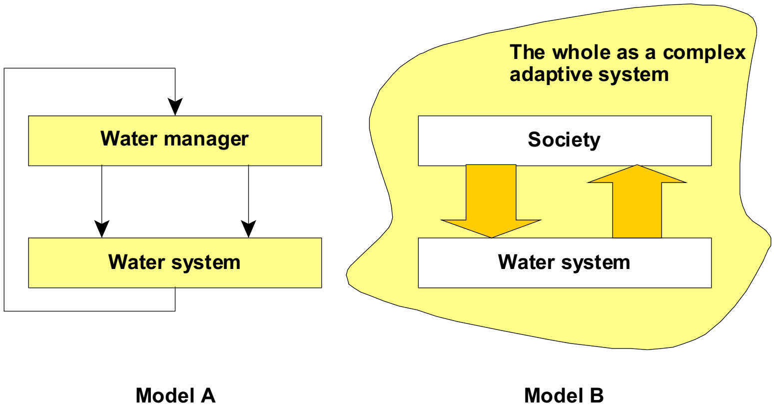

Figure 1. Schematic illustration of traditional (Model A) and integrated planning (Model B).

The traditional planning approach is adequate for isolated and well-defined systems, such as underground pipe networks and end-of-pipe treatment facilities. However, for water systems, where there is interaction between water and society, the traditional planning approach is not sufficient. To be able to handle the social dimensions in an appropriate way we must find alternatives to the traditional planning models. This means that integrated systems must be approached as complex adaptive systems with positive and negative feedback loops [2].

In the following the traditional planning approach ("Model A") and the new integrated approach ("Model B") will be discussed in more detail.

In Figure 1, Model A represents the traditional way of thinking. It assumes that the water manager controls the water system, so it will function properly. When the water managers detect deficiencies, they will compose an optimum package of corrective measures. The planning process in this model can be described in consecutive steps, which are passed through cyclically.

Model B represents a new way of thinking. The water system is regarded as an integrated part of the city, which displays a lot of dynamics. The model focuses on the interactions between the water system and society. These interactions are complex by nature, because many actors are involved in the process, a large variety of structures exist, and different scales and many policy fields are covered, like traffic, spatial planning and housing. It is also an interface between technical and social sciences. The whole of processes is assumed to be a complex adaptive system. The system has the ability to adapt its structure when the environment changes.

At first sight there seem to be only minor differences between the two planning approaches. However, in practice the differences are huge. Some of these differences will be explained here in more detail.

3. System complexity

Urban water systems, which interact with societal processes, are complex adaptive systems. The complexity that characterises such systems should not be disputed but be made manageable. Complexity must not be looked upon as something nasty that has to be reduced or avoided, but as a precondition for innovation and transition [2]. When a complex adaptive system is tamed and totally controlled, it loses its ability to adapt and to innovate.

Disputing complexity (as in Model A) means in practice that the integrated systems are reduced to such an extent that they show predictable behaviour that can be controlled. This means that only part of the system is in focus, i.e., the part that shows linear behaviour and which falls under negative feedback loops.

Managing complexity (as in Model B) means that we try to understand the value of the interwoven processes. The whole system then comes into view. This is when we discover that the system is not static but undergoes constant changes. In that, time plays an important role. With model B we illuminate subsystems with negative and positive feedback loops. Due to the positive feedback loops, processes accelerate and may show chaos, part of the time.

Complex adaptive systems exhibit patterns on which we can reflect during planning. In that, we have to strengthen the bond between the science and practice. Science offers new insights and tools for practice, and practice offers experience for science.

4. The role of time in planning

In complex adaptive systems time plays an important role [3]. Looking at planning processes in integrated urban water management, where insights and techniques have evolved dramatically in the last decades, we see that there has not really been appropriate importance assigned to time. Model A still dominates. In the following the importance of time will be discussed.

4.1 THE PAST

In systems with positive feedback the past does not fade away. A lot of values attached to water systems, especially the surface waters, have their roots in history. In plans we make with model A, the history of an area is not taken into account. We recognise the present and the future, where the future should offer a better state than the present. So – measures are planned. History does normally not influence the process of selecting appropriate measures.

History is of importance especially in water management, because in every country the water systems represent cultural values. In model B the history is taken into account and the connection between water and culture is examined. Experience shows that when source controls or open drainage concepts are applied in stormwater management, local residents will often respond enthusiastically, particularly if the cultural values are increased [4].

4.2 THE PRESENT AND THE FUTURE

In traditional planning, the difference between the present and the future is artificial in many cases. Goals are formulated for the future (e.g. "by 2010, emissions from the sewer system have to be reduced by 50%"). Later, when these goals have been reached, the state of the system will have improved. For that to happen, however, the goals have to be both rigid and verifiable. Today we often apply standards as goals.

How do we cope with the goals and standards in Model A? We compare the present state of a system to its desired future state. By doing that we suppress time and confront ourselves with a pile of bottlenecks – problems – in the present. Time has disappeared. To reduce these bottlenecks we identify measures. After that, we use complex planning strategies to prioritise the measures and reintroduce the eliminated time. The result is a measuring rod of time, with measures neatly organized alongside. This time structure is the result of a negotiation process, in which ambition and costs are important variables.

4.3 THE PERCEPTION OF TIME

Time is a rich concept. At the level of integrated water management plans, time manifests itself in a creative manner, i.e., as the carrier of change. Sometimes we can predict change, but often we cannot. Time produces surprises and if we do not like them we experience time as an obstacle and an enemy. It is an obstacle when it separates us from goals in the future and an enemy when it reveals our inability to control environmental problems. By means of advanced models we create the impression that complex processes can be predicted and controlled. When it does not work out the way we expect, we feel guilty or we blame someone else. A cyclic pattern of problem, solution, control, failure and guilt develops.

It is better to form an alliance with time (Model B) and try to appreciate the beauty of the uncontrollable. By trying to understand the uncontrollable, we can discover new patterns that can help us to navigate through a complex world. People who take their time do not experience it as an obstacle. However, time as an obstacle increases when activist groups force the local government to reduce time intervals. (e.g., "The goals must be met by 2008 instead of 2010!")

4.4 GOOD TIMING

Not all moments in time are suitable for implementing ideas for improving urban water systems. This is especially true in the planning of open water systems in the urban environment. It is an art to give planning processes the time they need. Visions about water systems and society must not be translated into rigid goals too quickly. Pressing the "turbo button" in order to achieve the goals must be avoided. Speeding up the planning processes often results in delay ("More haste, less speed"). Mumford [5] distinguishes two types of time: the chronological time and the kairological time (Kairos = the right moment). The former offers a structure for events and measures, whereas the latter emerges from knowledge, intuition and experience, and consists of "the right moments". Intervening in processes requires good timing. To have a feel for the right moment is important, because sometimes it is better not to act. Although this increases the complexity of interventions, it also makes them more realistic and effective.

5. Many opportunities lie in the unknown

When we explore in planning the relationships between goals and measures and between the cause and effect, we meet uncertainty. Sometimes it is possible to predict the effects of different interventions in the system. However, there are a lot of cases where the outcome could be a surprise. We can distinguish three degrees of uncertainty in the cause and effect relationships: certain, uncertain and structurally uncertain.

We talk about structural uncertainty [6] when it is impossible to reduce the uncertainty. For example, it is very hard to predict how people will react to an integrated water plan, as it is difficult to model public and political support. Furthermore, societal systems often show a deterministic chaos. Consequently, it is impossible to predict long-term future behaviour, even if we have a perfect model.

To avoid chaos and structural uncertainty, and reduce complexity (Model A), an appropriate strategy is often attached to the model. Interventions are focused on cause and effect relationships that can be well described and exhibit little uncertainty. In practice this means that only the processes in the biophysical water system are taken into account and the interactions between water and society are taken for granted. Society is in equilibrium. This strategy of avoiding uncertainty gives rise to concern, because the change will be incremental.

To really change system behaviour, the area of structural uncertainty offers the best prospects. The reason is that the associated processes have the highest degree of freedom. Many opportunities lie in the unknown. The future may reveal change that coincides with a shared vision on sustainable water management. For that, a strategy that fits Model B shows a healthy balance of uncertainty. The interactions between water system and society are taken into account and interventions are carried out in both the water system and the society. To cope with the structural uncertainty, the strategy includes learning by doing. Over the course of time, we learn.

6. Traditional planning

Storm drainage is the city's Drainage Department responsibility. As traditional stormwater facilities (pipes and detention tanks) are located underground, the planning, design and the construction is normally carried out without the involvement of any other technical department of the city (Model A). Figure 2 schematically illustrates the traditional planning of measures to upgrade an existing urban stormwater system.

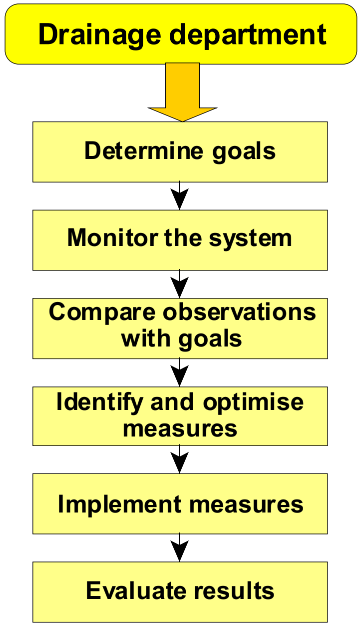

Figure 2. Illustration of traditional planning of stormwater facilities.

As a base for the planning a goal is set up in terms of runoff standards that are to be fulfilled. With the help of advanced hydraulic modelling tools alternative measures to improve the system are identified. A cost benefit analysis is carried out to select the optimal set of improvement measures. When the measures have been implemented the result is evaluated by monitoring the system. The typical feature of planning is that in practice it is fully controlled by the Drainage Department (Model A).

Model A is still the prevailing approach to planning measures in urban stormwater systems. This is all right as long as the measures are limited to the construction of facilities, which do not directly intervene in the urban environment. However, the introduction of sustainable stormwater drainage has changed the pre-requisites for the planning. Because the measures are integrated with the urban environment in quite another way than the traditional underground stormwater facilities, Model A is no longer adequate. The planning cannot be carried out by the Drainage Department alone, but must be carried out in co-operation with other city departments and with the involvement of the public.

7. Integrated planning

To succeed with the implementation of the concept of sustainable urban storm drainage it is of utmost importance to consider the societal aspects that are associated with the integration of stormwater facilities into the urban environment. Examples of these aspects are given in Figure 3.

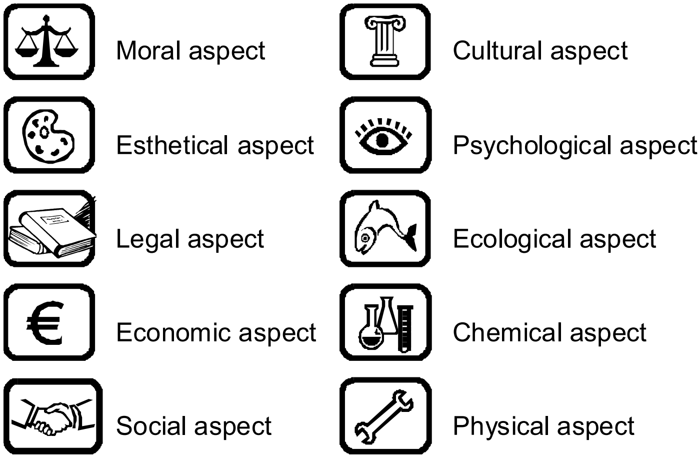

Figure 3. Aspects of water [7]

The introduction of sustainable stormwater drainage makes the planning much more complex (Model B) compared to the planning of traditional stormwater facilities (Model A). An active involvement of several city departments is needed. This involvement is seldom problem-free. Most cities do not have the tradition of cooperation in the planning and implementation of jointly owned and operated water facilities. The institutional barriers to such initiatives are often unexpectedly high. In the City of Malmö in Sweden it took many years to overcome these barriers.

During 10 years of experience with the sustainable stormwater drainage in the City of Malmö a new planning approach for this type of infrastructure has evolved. In Figure 4 this approach is outlined for a typical example of integrated planning (Model B).

When sustainable stormwater management is applied in Malmö, typically, at least three departments in the city administration take active part in the planning process. One important element in the planning is the involvement of citizens, schools, and the special interest groups, which can have interest in the planned facilities.

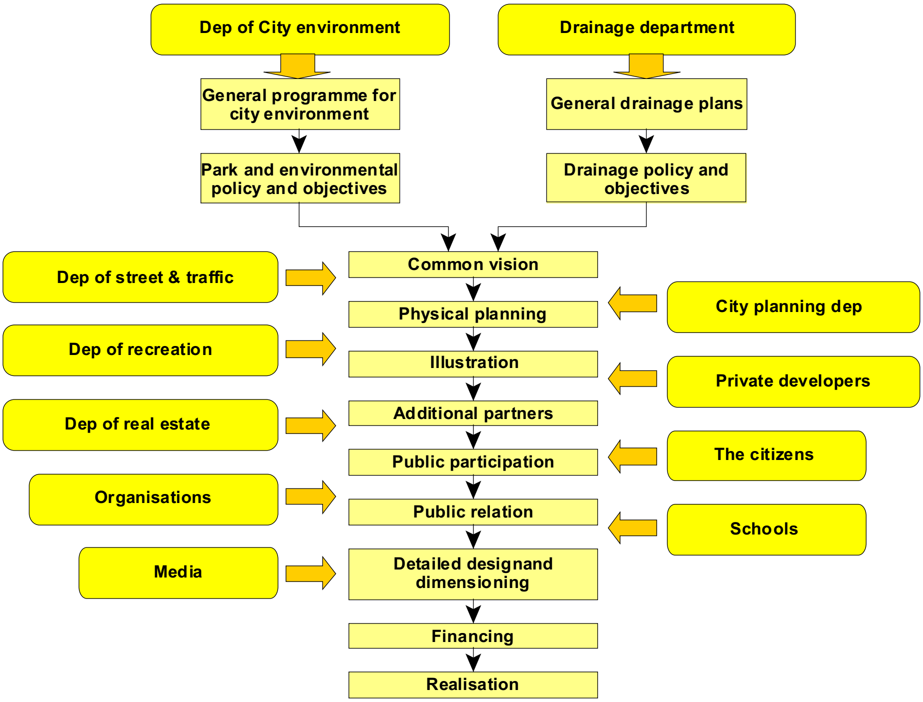

Figure 4. Schematic illustration of integrated stormwater planning in Malmö (Model B).

The departments of Drainage and City Environment develop their common vision together with representatives from the City Planning department. Depending on the nature of the suggested drainage facilities other city departments are involved in discussions of the implementation of the vision. If for example the suggested facilities are to handle polluted runoff from roads and other traffic areas the Street department is also involved in the discussions. If the facilities are to be used for recreational purposes the Department of Recreation is involved.

At the earliest possible stage the developed vision must be introduced into the city's physical planning process. In practice, the concept of sustainable stormwater drainage can often become the driving force for developing the plans. Potential additional partners are approached in order to broaden the economic base for the implementation. Private developers and the city's own real estate department are partners that might see a benefit in taking part in the project. One incentive for them could be that the prize of the land might increase, if a "blue-green" corridor including open water is introduced in or around the development.

It is important to achieve public participation in the planning process. The visions should therefore be introduced, at the earliest possible stage, to citizens, schools and other special interest groups in the area. A humble attitude towards the public demands and requests will facilitate the public acceptance of the facility. Local media can play an important role in the promotion of the planned facilities.

8. Interactive Implementation

The order in which actions take place has changed in Malmö. Time plays a different role. In former days a serial approach was applied. Based on policy papers a plan was made. This plan presented optimised measures, founded on intensive model studies. Of course the models could only focus on the stormwater and sewer processes, not on other processes in the urban areas. When the plan was accepted and the financing was well organised, the drainage department made the designs and implemented the construction. After that, the facilities were handed over to the people who do the maintenance.

When the focus is restricted to sewer management only, this serial approach (Model A) suffices. However, for integrated planning it shows many shortcomings. It is impossible to derive all goals from policy papers to make an ideal plan without uncertainties. Urban dynamics takes place at different time and space scales and it is far too complicated to connect them all. Also, the arena of people involved in the process of planning and design differs from the implementation arena significantly. The maintenance people do not interact with the designers. Reducing the costs of a design often results in higher maintenance costs. These higher costs are not included in the plan.

The serial approach shows characteristics of a relay race where the baton is handed over from one runner to another, in a strict order of time, and as quickly as possible. Especially the process of passing the baton is crucial and very critical.

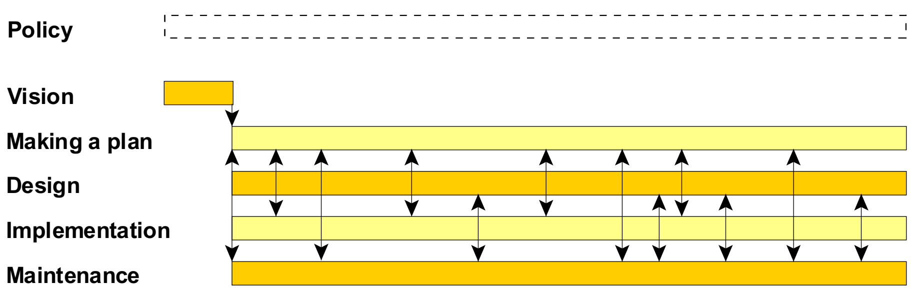

Figure 5. The parallel approach to Interactive Implementation (Model B).

The practice in Malmö shows a different approach, as displayed in Figure 5. This approach is called Interactive Implementation [7]. In interaction with a lot of actors, a vision is developed for the stormwater management, related to other societal processes in the urban areas. This vision is not a detailed plan, but an attractive presentation of the stormwater system in the future, with elements at several time and space scales. This vision is easy to communicate and makes people enthusiastic, like citizens, politicians and people from other municipal departments. After that, the plan preparation process, design, implementation and maintenance are handled in parallel. When there is the right moment (Kairos), parts of the vision are brought into construction. For other parts, model studies are carried out. Maintenance people are involved in the design processes. Creativity is not restricted to the design process, but new insights can be acquired also during construction.

Interactive implementation shows the characteristics of a learning process, where people act in parallel and learn from each other's mistakes. Practice and science are interwoven [8]. Parts of the vision are constructed, even when the financing of the whole vision is not fully organised. Most of the public and political support emerges out of the process when practical examples are shown like in Augustenborg (next chapter). Spatial integration takes place on a large scale (the vision) and integration of water, traffic, recreation, education and culture takes place in small-scale practical projects. In this, the system complexity is not disputed, but has been made manageable.

9. Sustainable storm drainage – examples from Malmö, Sweden

9.1 GREEN ROOFS IN AUGUSTENBORG

The residential area Augustenborg in Malmö [9] is serviced by a combined sewer system. The area has suffered from frequent basement flooding. In order to reduce the risk of flooding different source control techniques have been introduced. The primary goal of these was to reduce the stormwater runoff to the overloaded combined sewer system.

Figure 6. A section of the green roof installation at Augustenborg in Malmö.

In the Augustenborg area there is a site with an engineering workshop and a garage. Most buildings here have flat roofs. To reduce the stormwater runoff from these roofs it was decided to apply a vegetation cover of sedum grass on top of 9,500 square metres (0.95 hectares) of the roofs. An evaluation of the structural strength of the existing roofs showed that no extra reinforcement was needed to carry the extra load.

The green roof installation was designed as a full-scale research facility, where the effect of different grass species, thickness of the soil substrates, the slope of the roof, etc. could be investigated. Figure 6 shows one section of the green roof installation in Augustenborg.

The main reason for applying green roofs was their capability to reduce the stormwater runoff from the roofs. Among the other advantages with this technique can be mentioned that the green roof serve as an effective insulation of the building and thus contributes to savings of heating energy. In addition the green roof protects the roof structure. Calculations show that the lifetime of the roof construction can be prolonged quite considerably.

9.2 OPEN DRAINAGE IN A PARK IN AUGUSTENBORG

To reduce the speed of stormwater runoff, an open drainage system was constructed through the residential area of Augustenborg. In an existing park the open drainage system was designed in the form of a shallow drainage ditch (swale), which is fed with water only during wet-weather conditions. During dry weather conditions the drainage ditch is totally dry. One advantage of choosing an open drainage ditch was that the construction cost was very low and that the ditch could be easily integrated into the park environment. The new drainage ditch in the park is shown in Figure 7.

Figure 7. Open drainage ditch in the existing park in Augustenborg

The idea of constructing an open drainage ditch originally came from the residents within the area. Some of them had experienced an old open creek in the park, which was covered when the area was developed several decades ago. No doubt the link to the “historic” creek through the area was very much appreciated by the residents. In addition the City Environment Department considered that the new open drainage ditch increased the aesthetic value of the park.

9.3 DETENTION OF THE STORMWATER RUNOFF FROM THE SCHOOLYARD IN AUGUSTENBORG.

The schoolyard in a residential area of Augustenborg is highly impervious (roofs and pavements). Almost all runoff from the schoolyard was connected to the combined sewer system. As part of upgrading the sewer system, the impervious surfaces were remodelled so that most of the runoff was diverted directly to the new open drainage system in the area.

Figure 8. Detention basin in the form of an amphitheatre in the schoolyard of Augustenborg

To slow down the peak flows of runoff, an open detention basin was constructed in the schoolyard. After discussions with the school authorities it was decided to design the basin in the form of an open amphitheatre (see Figure 8). The goal was to be able to use the basin for outdoor lectures.

9.4 STORMWATER DETENTION IN THE HUSIE LAKE

To handle stormwater from future settlements in the eastern parts of Malmö large detention volumes were needed. The closing down of an old military training field made it possible to create a new lake, the so called Husie Lake. In historic times the area of the new lake had been a wetland, which however over the last hundred years had been effectively drained.

The new Husie Lake was created in natural lowland and was designed with meandering channels, wetlands, small islands and even one small waterfall. The excavated material was used for landscaping the area around the lake. The lake has a size of about 4 hectares (10 acres) with a maximum water depth of 2 metres (6 feet). The lake will receive stormwater from future settlements in the area as well as drainage water from existing agricultural land.

The area around the lake has become an attractive recreational area for citizens in the whole eastern part of Malmö. Surrounding schools and Kindergartens use the lake for educational purposes (see Figure 9).

Figure 9. A group from a Kindergarten is playing at the outlet of the Husie Lake

10. Conclusions

The introduction of the concept of sustainable stormwater drainage has led to a new approach in which stormwater drainage is no longer looked upon just as a technical service that is supplied by the city's Drainage Department. Stormwater has become a positive resource in the urban environment for the citizens. Under such circumstances, many city departments must take more active part in the stormwater management planning.

A close co-operation among the different technical departments in the city and an active involvement of the public has proved to be of utmost importance for a successful implementation of the concept of sustainable stormwater management. This integrated approach (Model B) is much more complex and time-consuming than the traditional approach to stormwater planning (Model A).

Many cities tend to continue to use the traditional approach (Model A) for planning sustainable stormwater solutions. As this approach is not very well suited for planning stormwater solutions, which are closely interacting with the societal processes in the urban environment, the result will often be unsuccessful. For planning sustainable stormwater systems one should apply Interactive Implementation (Model B).

Experience shows that sustainable stormwater drainage is a most efficient way of handling stormwater from both existing and new developments. It must be emphasised that time plays an important role in the integrated planning. To obtain public and political support for a plan, it is necessary to include among others history, timing and uncertainty in the planning process.

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