Concrete In The Service Of Mankind

Appropriate Concrete Technology

Edited by
RAVINDRA K. DHIR
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Concrete
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Appropriate Concrete Technology

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The cover illustration shows the Glendevon Reservoir spillway, constructed using GGBS concrete. Photograph courtesy Castle Cement Ltd.

PREFACE

Concrete is ubiquitous and unique. Indeed, there are no alternatives to concrete as a volume construction material. This raises important questions of how concrete should be designed and constructed for cost effective use in the short and long-term, and yet encourage further radical development. Equally, it must also be environmentally-friendly during manufacture and in its aesthetic presentation in structures.

The Concrete Technology Unit (CTU) of the University of Dundee has organised this major 5 day International Congress, following the conferences, Protection of Concrete in 1990 and Concrete 2000: Economic and Durable Construction Though Excellence in 1993, as part of its continuing commitment to the development of excellence in concrete construction.

The central theme of the Congress was Concrete in the Service of Mankind, under which 5 self-contained conferences were organised; *(i) Concrete for Environment Enhancement, (ii) Concrete for Infrastructure and Utilities, (iii) Appropriate Concrete Technology, (iv) Radical Concrete Technology* and *(v) Concrete Repair*, *Rehabilitation and Protection*. In total 350 papers were presented by authors from 70 countries worldwide.

The Congress Opening Addresses were given by the Lord James Douglas-Hamilton MP, Minister of State for the Construction Industry, Scotland and by Dr Ian J.Graham-Bryce, Principal and Vice-Chancellor of the University of Dundee. The Opening Papers were presented by Emeritus Professors P.Kumar Metha and Ben C.Gerwick, University of California, Berkeley, USA and Professor John Morris, University of Witwatersrand and Mr Spencer S.Sephton, PPC Cement (pty), South Africa. The closing address was given by Professor Peter C.Hewlett, Director of the British Board of Agrément, UK and Visiting Industrial Professor, Department of Civil Engineering, University of Dundee.

The Congress was supported by 14 major International Institutions together with 23 Sponsors and 50 Exhibitors, highlighting the importance of concrete and the close cooperation between the CTU and industry.

A Congress of this size and scope was a major undertaking. The immense efforts of the Organising, International Advisory and National Technical Committees, who advised on the selection and review of papers is gratefully noted. The efforts of all the Authors and Chairmen of the various Technical Sessions and, in particular, those who travelled from afar to come to Dundee are greatly appreciated as are all the CTU staff and research students for their sterling efforts in ensuring the smooth running of the Congress. Particular thanks must be given to the two Congress joint Secretaries, Mr Neil A.Henderson and Dr Michael J.McCarthy and the Unit Secretaries Mr Steven Scott and Miss Diane Sherriff.

All the Proceedings have been prepared directly from the manuscripts provided by the authors and, therefore, there may be some errors or inaccuracies that have been inadvertently overlooked.

Ravindra K Dhir

Dundee
February 1996

INTRODUCTION

The specification of concrete has long been a major source of conflict between the engineer, contractor and supplier. Much of this conflict stems from the increasing litigious world in which modern construction takes place. Equally, it may arguably to be due to the misunderstandings arising from what is the appropriate performance that will be required from the materials used themselves and the structure manufactured.

To overcome this conflicting interest, there will have to be an evolutionary approach away from the traditional method of specification and towards performance criteria. This will also facilitate the formulation of integrated, harmonised international standards.

This transition will not be easy and it will almost certainly transfer responsibility for some concrete material decisions to the supplier. It can only be hoped that total quality management, which has done so much for the manufacturing sector, can also provide the same degree of confidence to clients in concrete construction. Since most international specifications bodies are, in principle, committed to this approach to specification by performance, it is important that the implications of this are debated at a major forum such as this international conference.

The versatility of concrete continues to be underlined through the development of new binder technologies, alternative materials for reinforcement and novel design and construction techniques, which continues apace.

The Proceeding of this Conference *'Appropriate Concrete Technology'* deal with all of these subject areas under six clearly identified themes: (i) Criteria for Appropriateness, (ii) Implications of Harmonisation, (iii) Versatility of Concrete, (iv) Binder Technology, (v) Non-Ferrous Reinforcement and (vi) Design and Construction. Each theme started with a Leader Paper presented by the foremost exponents in their respective fields. In total, 65 paper were presented during the 3 day International Conference and compiled into these Proceedings.

Ravindra K Dhir
Michael J McCarthy

Dundee
February 1996

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THEME 5 NON-FERROUS REINFORCEMENT

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Leader Paper

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Theme 1
CRITERIA FOR APPROPRIATENESS

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Leader Paper

Performance Criteria for Structural Concrete

Professor G Somerville

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PERFORMANCE CRITERIA FOR STRUCTURAL CONCRETE

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UK

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ABSTRACT. A proposal is made for a framework for durability design, which is potentially numerate and compatible with conventional structural design. It is strongly suggested that the definition of limiting performance is an essential first step, to permit sensible economic/technical decisions to be taken, when designing with life cycle costing in mind. Follow up action is then required to similarly evaluate the available resistance options on a common basis.

Keywords: Durability, Performance criteria, Concrete, Design, Construction, Materials.

Professor George Somerville is Director of Engineering at the British Cement Association. He is also a Visiting Professor, both at Imperial College, London, and at Kingston University. He is a structural engineer, with strong research interests in service life performance, including durability design and assessment.

INTRODUCTION

As a student in the 1950s, the author was taught the rudiments of structural design in terms of using numerical methods to provide adequate strength, stiffness, stability and serviceability—with some guidance on how to do that at least first cost. The performance criteria, margins, and factors of safety were given in Codes, and accepted without question. These were assumed to prevail as soon as the structure was built, as well as during its entire useful life (although this was never specified). This was not checked in any numerical sense, but assumed to be so, if material and workmanship specifications were satisfied during the construction phase. In short, the provision of durability i.e. the maintenance of the required margins and factors, was on a purely prescriptive basis.

In the 1990s, that basic scenario has not really changed. To be sure, the prescriptions, specifications and factors of safety have altered, but the overall design approach is fundamentally the same.

Feedback from performance in service has indicated that durability is a major problem, and is therefore the principal reason for the changes made in concrete specifications. Specific aggressive actions have been identified and quantified, with provision made to deal with these on a prescriptive basis. A simplistic summary of these developments is given in Table 1. For those actions which predominantly affect the concrete, the prescriptive approach is perhaps best; problems only really arise when several aggressive actions occur simultaneously. For corrosion—the major durability issue—the matter is not so straightforward. Much work has been done in increasing the understanding of the corrosion process, and many proposals made for improving durability, which fall into one of two categories:

(a) increasing cover and/or changing the concrete mix, in terms of ingredients or proportions, or
(b) introducing protective systems in the form of coatings, sealers, penetrants or cathodic protection.

There is little doubt that these proposals give better durability. How much better may still be in question in some cases, since field experience is relatively short; how they might complement each other is yet another question.

Some 10 years ago, the author [1] made a distinction between the production and placing of durable concrete and the design and construction of structures that will be durable. The essence of the argument was that the industry tried to solve all its durability problems via a prescriptive materials approach, and yet standards of design, detailing and construction were at least as significant. The trend of introducing protective systems tends to move durability considerations out of the materials court into the design arena. Another major factor is the growing awareness of the need to consider management and maintenance even at the design stage and of the need to attempt this via life cycle costing [2].

For any quantitative durability design approach to work, it is essential to define performance requirements, both in terms of time and of minimum technical performance.

Table 1 Conditions requiring special attention in durability terms

CATEGORY			**COMMENTS**
(1) Those prodominantly affecting the concrete	(a) Freezing and thawing		Usually dealt with by: - suitable choice of materials, mix proportions and concrete grade - designing to minimise exposure to moisture - air-entrainment; air content depends primarily on aggregate size - adequate curing and compaction Note that the use of deicing salts can aggravate the problem.
	(b)	(i) Sulfate	Fairly well understood.

	Aggressive chemical exposure	attack	Specific material and mix proportions are recommended for clearly defined ranges of sulfate concentration.
		(ii) Acid attack	Can come from various sources, e.g. ground water, sewage, farms, industrial processes. pH value is a guide to severity (below pH=4.5 special protective measures usually necessary). Low permeability concrete will provide acceptable protection against mild attack.
	(d) Chemical reaction of aggregates	(i) e.g. Alkali-silica reaction	Unusual, in arising from the basic materials in concrete, rather than from ecternal attack. The basic reaction is fairly well understood and provisions exist to minimise the risk of damage due to the reaction—although all potentially reactive aggregates have probably not yet been identified at a European level.
(2) Those predominantly affecting the reinforcement	(a) Corrosion	(i) Due to carbonation of the concrete	Corrosion is a consequence of the penetration of liquids and gases through concrete to reach the reinforcement: the processes involved are now well understood. Resistance to these actions depends fundamentally on the permeability of the concrete, and hence on the four 'Cs' [Constituents (of the mix), Cover Compaction, Curing].
		(ii) Due to diffusion of chlorides	There are two distinct phases in the mechanism of attack—the time taken for the deleterious substances to reach the steel, and then the rate of corrosion. Both periods require estimation in assessing loss of serviceability and hence design life. These may be different for different forms of attack, and hence the need to distinguish between, say, carbonation and chloride penetration. Current design approaches, under development, are bases fundamentally on this 2-phase mechanism.
		(iii) Other potential harmful gasses or liquids	There are various options available to the designer in providing adequate resistance, including surfact protection of the structure, direct coating of the reinforcement, cathodic protection, adjustment to the 4 'Cs', etc. The option chosen depends on the severity of the attack and on economic factors.

This is no different from establishing minimum requirements for safety and (say) crack width, in structural design; without such metre-sticks, it is not possible to compare alternative options in whole life cost/benefit terms.

In performance criteria terms, durability is not the only issue. At present in the UK, the construction industry is itself undergoing change, in the procurement process. For some time now, concrete technology has had to cope with large continuous pours, with pumping, and with round-the-clock production in a precast factor, and with delivery in a ready-mix truck. However, demand for increased productivity and quality, linked to speed of construction, is imposing increased pressure on technology for the construction phase. Design is becoming more closely related to the construction process, and therefore we have to strike a balance between buildability and satisfactory performance in service. This means that performance criteria must be derived to permit that to happen, i.e. to balance the needs both of buildability and durability; the merits of alternative design options can then be judged on a common basis.

In brief, the time has come when durability should be considered as an integral part of structural design—ideally based on the same principle of establishing margins or factors of safety, in providing resistance against known loads, while satisfying established performance criteria.

The purpose of this paper is to propose a framework for such a design method, before going on to suggest some limiting performance criteria.

WHY AN ENGINEERING APPROACH TO DURABILITY?

Data of the type provided by Paterson [3] and Wallbank [4] clearly demonstrate that the level of deterioration in individual cases is dictated by a combination of factors, in which design and construction issues are significant. The provision of durable concrete is important, but it is possible to have 'failure' even with the best materials, if design and construction standards are poor.

Moreover, the spectre of obsolescence also enters the arena. Many buildings and bridges have had to be upgraded or replaced, because their functional needs have changed—quite apart from any decrease in technical performance. In addition, different components in individual artifacts (eg. cladding in buildings; expansion joints in bridges) have been shown to have useful lives much less that those for the basic structures.

Growing recognition of this situation has led to the development of a performance profile approach to design, eg. by White [5], for buildings, and by BSI [6] in a formalised general approach. It is not always possible to precisely define the useful life of a structure at the design stage or to second-guess the need for future upgrading. However, in many cases an informed decision can be taken; for example, in the UK, much of the motorway network is being upgraded within 30–40 years of construction.

This leads to a basic proportion as follows:- durability should be an integral part of design: the essential requirement is to ensure fitness for purpose, while taking account, as far as possible, of future functional and financial needs, in whole life costing terms.

ELEMENTS IN A DESIGN FRAMEWORK

Table 2 shows the five essential elements which make up the detailed process of structural design. To gain acceptability, durability design has to be seen alongside these, and parallel proposals are also made in Table 2. Currently we concentrate on item 5, while trying to improve, via research mainly associated with items 1 and 3. Missing is any serious consideration of items 2 and 4, and yet both are fundamental to a quantitative approach; we need to know what we are trying to achieve, in what will never be an exact science, and we need to have some confidence in the predictions we make.

To understand better what is intended here, Figure 1 is proposed as a possible relationship between the elements in Table 2. Each aspect of Figure 1 will now be considered in more detail.

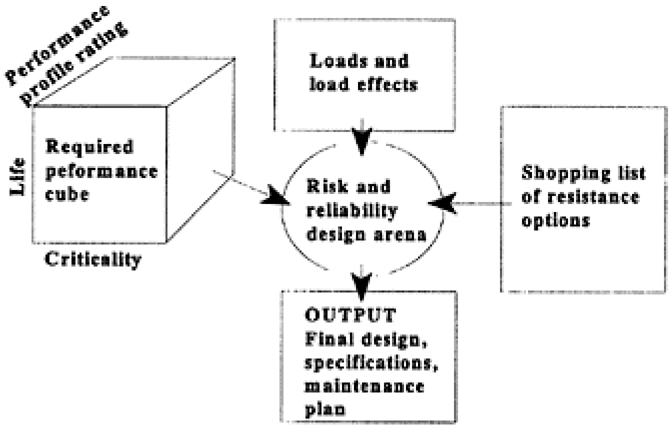

Figure 1 Suggested outline relationship between the elements in a durability design framework

LOADS AND LOAD EFFECTS

In designing for durability, a knowledge of the 'loads' is more important than in structural design. How the effects of these loads are treated will depend on the level of calculation proposed in the design strategy.

The situation is summarised in Table 3. As a minimum, Zone A in this Table will always be necessary, particularly if going for a prescriptive type of solution. However, it is strongly suggested that, to be truly effective, the definition of exposure conditions has to be orientated towards specific deterioration mechanisms: this is contrary to the present system in the UK, but consistent with the proposals in the European Standard for concrete, ENV206.

Table 2 Elements in a design framework

ITEM	ACTUAL FOR STRUCTURAL DESIGN	PARALLEL POSSIBILITIES DURABILITY DESIGN
1. Loads	Imposed loads taken from Codes.	Classification of environments.

		Identification and quantification of aggressive actions.
2. Performance Criteria	Adequate strength, stiffness and serviceability. Deflection and crack width limits.	A statement of required life in qualitative of quantitative terms. Some account of criticality (risk analysis). A definition of a performance profile including any strategy for maintenance. Specific limits to 'damage' or effects of deterioration (e.g. cracking or loss of section due to corrosion; expansion due to ASR; internal damage due to freezethaw.)
3. Modelling Analysis	Methods of analysis used to determine action effects due to the applied loads. Design equations used to provide resistance to the action effects (bending, shear, etc.). Recommendations on detailing.	Predictive models to determine the effects of the aggressive actions. Models/equations used to calculate the effects of deterioration on conventional action effects. Evaluation of alternative options in providing the required resistance for the required time—usually a combination of material, design and construction options.
4. Factors of Safety, Margins	Partial factors which are generally applied both to the loading and resistance sides of the design condition $S \leq R$. Can also be built in to design equations. Margins may be set, in establishing limiting performance criteria (e.g. crack widths).	Ideally, should require the same approach as for structural design. Should be done consciously, depending on knowledge of loads, required life, risk analysis, precision of models, etc, etc.
5. Specifications, Certifications, QA	Concrete mix ingredients and proportions. Cover. Rebar Specifications. Minimum workmanship requirements. Supporting certification and QA schemes.	Essentially the same as for structural design, but with more factors included (e.g. coatings, special steels, cathodic protection) and possibly more options (e.g. different classes of construction). Supporting certification and QA schemes may have to be stronger.

Table 3 Durability loads and load effects—treatment in design

A	1.	Identification of aggressive action by type and intensity.
	2.	Definition of the outer environment by broad category (for level 1 'prescriptive' design solutions).
B	3.	Predictions of inner environment, where appropriate, and definition of most severe conditions for individual aggressive actions or relevant combinations of these.
	4.	Modelling of deterioration processes, to provide service life calculations on a probabilistic basis, to meet agreed performance criteria (taken from the performance cube—Figure 2). Modelling may have to be 2-phase: (i) predicting deterioration itself, and (ii) predicting the

effects of deterioration on resistance (bending, shear, bond).

practice indicate that ponding and run down of water is probably more severe than spray, but that the process of wetting and drying can override that.

Table 3 also stresses the need not only to predict deterioration itself but also to assess the possible effects on structural resistance. Some structural forms are more sensitive to the effects of deterioration than others, and the risk factor requires evaluation.

THE REQUIRED PERFORMANCE CUBE

A proposal for this is shown in Figure 2. The front face of the cube is used to make a statement about the importance of the structure or element, in terms of 'life' and 'criticality'. While there has been discussion in the literature regarding specifying design lives for different types of structure in years, this may not be necessary. The proposal here is that zones be defined (A-G in the Figure), which set down performance requirements in terms of life and criticality. Criticality is intended to recognise both the importance of structural elements in load-carrying terms, and also the difficulty of repair or replacement, and the disruption that this would cause. A possible grading for criticality might be:

- High : failure would cause cessation of function and/or major disruption during remedial work.
- Medium : efficiency of operation would be reduced, but replacement/remedial work could be done out of normal working hours.
- Low : not critical. Any necessary maintenance or remedial work could be done without inconvenience.

The grading is similar to the British Standard idea of classifying components as replaceable, maintainable or lifelong [6]. Any foundation or key structural element might be categorized as 'high criticality' or 'lifelong', for example.

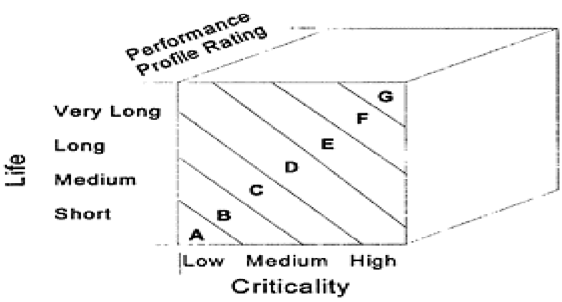

Figure 2 The required performance cube

If necessary, life could be defined in years, but the zonal approach forces designers to take early decisions on maintenance/replacement strategy, related to perceived future functional requirements. It also helps define limiting performance criteria for durability. This brings us to the third axis of the cube—the performance profile rating.

Rigorous design for durability effectively requires the definition of appropriate performance criteria for each aggressive action—equivalent to maximum deflections or crack widths in structural design. The intention can be illustrated by considering corrosion; possible limiting criteria are:

1. The mean maximum level of a carbonation or chloride front should not penetrate more than (a)% of the nominal cover.
2. Corrosion has just started, i.e. the critical front has reached the reinforcement, both oxygen and water are available, and, in the case of chlorides, the critical chloride threshold level has been reached.
3. Corrosion has caused cracking parallel to the reinforcement and the crack width is equal to (b)mm.
4. Corrosion has removed (c)% of the cross-section from (d)% of the reinforcement.

If these are then related to Figure 2, then examples of limiting criteria which might then emerge are:-

• Zone A : mean maximum level of a chloride profile should not reach the rebar for [15 years]; loss of rebar section (<10%) after [30 years].
• Zone G: mean maximum level of the chloride profile should not reach the rebar for [100 years].

These examples require very different design solutions. The concern at this point is with performance criteria for structural elements. The objective is to create flexibility, whereby the designer can consciously make a choice commensurate with his structure and its maintenance/replacement strategy. In effect, this approach is an extension of methods already in use for assessment work, which involve the definition of a range of damage levels.

Having established limiting criteria, the next question is how can a designer ensure that these are satisfied. One obvious direct approach is via Zone B in Table 3. For one-off special structures, this may be the best way forward. However, for general use, there is an alternative, which is outlined below.

SHOPPING LIST OF RESISTANCE OPTIONS

In current activities aimed at improving durability standards for grouted post-tensioned bridges, Raiss [7] has proposed the concept of multi-layer protection, in consciously designing for durability. In effect, this recognises that any single method of protection may fail or wear out, and there is a need for redundancy in the design system. This approach is capable of extension to other applications and hence the proposal here of developing a shopping list of options from which to choose.

An embryo shopping list is given in Table 4 for corrosion. Note that the Table is divided into three zones, A, B and C—broadly corresponding to material, design and construction matters, and there probably would have to be minimum requirements specified from each zone. Not all of the options are equal by any means, and there would be a need for development work to evaluate their relative merits.

Ideally, each option should have a performance profile, perhaps defined by a numerical rating. A summation of ratings could then be matched against defined requirements for each zone in the required performance cube (Figure 2)—say 10 units for Zone A and 100 units for Zone G.

Table 4 brings material, design and construction options together. Almost all current developments are in the materials sector, while attempting to produce greater longevity is an open-ended way, while virtually ignoring the design and construction issues. If targets are set (Figure 2), then the current R & D effort could be focussed on developing Table 4, in cost-benefit terms on a whole life basis.

RISK AND RELIABILITY DESIGN ARENA

This is illustrated in Figure 3. The actions foreseen are in the diagram itself, and the Notes indicate how the system would work.

The whole emphasis in Figure 3 is still on durability design. However, this would have to be considered alongside the needs of conventional structural design (Table 2) and of buildability. As an example, consider a bridge using precast standard bridge beams. For load bearing and prestressing, probably a Grade 60 concrete would be required. For rapid production, a high early strength would also be required. The structure could be in Zones F or G in Figure 2, with a consequential requirement that the chloride profile should not reach the tendons for (say) 100 years. These basic needs then establish a foothold in Table 4, and, assuming that reliable performance profiles are available, a choice can be made from various resistance options, to achieve the required performance.

Table 4 Shopping list of resistance options for corrosion

A Materials	Concrete quality - mix proportions - mix ingredients
	Cover - minimum - tolerances
	Permeable formwork Concrete protection - sealers
	- coatings
	- penetrants
	- layers
	Rebar protection - epoxy coating Special steels
	Non-corrodible reinforcement Cathodic protection
B Design	Design concept Structural detailing Cladding, services, fittings, finishes

	Articulation, joints, movement Treatment of water, drainage Control of the environment, barriers Provision for inspection, maintenance, replacement Accurate assessment of effects of deterioration mechanisms
C Construction	Construction methods Quality control Certification Testing Rationalisation, standardisation, simplification

CONCLUDING REMARKS

A proposal is made for a framework for durability design. It is not a ready-made system, but is put forward at this time to stimulate discussion, and to give a sense of direction to all the current R & D work on the different facets of durability.

The emphasis is on design, while taking account of material and construction matters. Design is not synonymous with analysis, and will never be an exact science. However, by properly defining performance criteria (both in breadth and in depth), then the strength of predictive modelling can be harnessed, to produce structures which better meet our needs, in life cycle terms.

Figure 3 The risk and reliability design arena

Notes to Figure 3

1. In the overall framework, at least 3 possible levels of design are foreseen:

   (a) A recipe approach, similar to current practice.
   (b) A more rigorous numerate approach, as briefly outlined in the text to Table 4.
   (c) An analytical approach, as briefly mentioned in zone B of Table 3, while meeting the performance requirements of Figure 2, and supported by the shopping list in Table 4.

2. It is important that all possible aggressive actions are identified and treated. Some, such as sulfate attack or abrasion can be dealt with purely by a recipe approach. However, others, such as corrosion and freeze-thaw, may produce synergetic effects.
3. Conventional structural design is on a semi-probabilistic basis. This suggests that durability design should be on the same basis; however, support should also be provided by Risk Analysis.

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WHO LIKES CONCRETE?

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Appropriate Concrete Technology. Edited by R K Dhir and M J McCarthy. Published in 1996 by E & FN Spon, 2–6 Boundary Row, London SE1 8HN, UK. ISBN 0419 21470 4.

ABSTRACT. People live with concrete. The paper deals with the public perception of concrete, particularly as revealed by the misuse of the word itself. A survey of people's attitudes to concrete is described briefly. The problems in achieving satisfactory concrete surfaces are outlined. The author describes a new serviceability limit state of appearance which might be an answer to the vexed question of what is an appropriate surface in a given situation.

Keywords: Concrete, Appearance, Perception, Public, Brutal, Finish, Weathering, Limit state.

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INTRODUCTION

Concrete in the service of mankind presupposes that mankind wants to be serviced by concrete. Whether it does or not, the fact is that mankind has been "fed" with concrete for many years and it is inevitable that it will continue to be so fed for many years to come. But, behind the widespread use of concrete and behind all the major developments in improving concrete strength, addressing durability issues and understanding this most complex material, lurks the question—who likes concrete anyway?

Concrete is the bread and butter of many people. It is one of the major construction materials today. But the point is, how well do we know what the public thinks of "concrete" and perhaps more importantly, how well is concrete "sold" to the public?

A person's appreciation of a building is a very subjective thing, and also a very complex thing. When somebody says, on being shown a new building, "I like it" or "I like it not", the answer represents an almost instantaneous verdict on what is seen. It encompasses a wide range of simultaneous subconscious observations of shape, size, colour, surface appearances, shadows, relations with neighbouring buildings, interaction with the environment, etc., as well also as the observer's own preconceived ideas.. Concrete, for better or worse, is part of this building scene.

The appearance of concrete on the day of its execution is however, only a part of the story. The weathering of concrete on a long timescale is of no lesser importance. When the architect and the engineer and the contractor have long left the building, the concrete is still there, battling it out with the elements, internal as well as external. The rather poor appearance of a lot of concrete surfaces after several years of weathering has not helped the public image of concrete as a visual material. The interaction of the multiplicity of factors that affect a building over time is not fully understood or even appreciated by many involved in the final realisation of a designer's dream.

THE PUBLIC AND CONCRETE

The Popular Perception of Concrete

A perusal of the popular press is revealing insofar as the use of the term *concrete* is concerned. One can perhaps forgive writers of letters to editors for misusing the term to conjure up in another reader's mind some nasty association or to lend some extra emphasis to an argument. Concrete is perceived by such writers to have an inherent nastiness about it. There is less excuse for established feature writers to use the word *concrete* in a wrong way. After all, they are expected to rise above popular perceptions and to seek to demythologise common but erroneous perceptions.

The following examples have been taken from a recent selection of Irish newspapers. The authors shall remain nameless since the object of the exercise is to select randomly from the press with the aim of demonstrating the point made earlier. The bridges of the Alps are masterpieces of engineering and architecture, yet in an article concerning juggernauts, it was stated that: "The *concrete* bridges that carry the autobahns are an ugly visual intrusion, and the noise of the heavy traffic reverberates along the narrow valleys." (1). It would appear that the traffic noise was the source of disquiet, the use of ugly *"concrete"* bridges helped to make the point but concrete had really nothing to do with it. In Ireland, some interpretative centres have come in for some public criticism. To emphasise their perceived awfulness, a correspondent has referred to ".....plans to graft on to some of the most precious and vulnerable areas of the Irish landscape carbuncle-like *concrete* structures called interpretative centres." The point was made for the undiscerning reader, with the help of "concrete carbuncles", that interpretative centres were undesirable. (2). No further critical appraisal was apparently deemed necessary.

In describing the life of Ms. Masako Owada, before her wedding to Crown Prince Naruhito of Japan, it was stated by a correspondent that until her marriage, she was a "dweller in a *concrete* house in a Tokyo suburb…." (3). Perhaps she was, but her rise from a humble existence was the apparently expected implication by the reference to a *"concrete"*, and ipso facto, humble home.

There are many other examples. A feature writer, referring to a housing estate, wrote "it is easy to see how, in the damp treeless wasteland of *concrete* wall and exterior plumbing, inertia can sap all but the most energetic." (4). Elsewhere a writer, referring to families moving to the country, wrote "city folk embarked on the intrepid transition to fresh air and country living, far from the *concrete* and grime". (5). There is the subtle association of concrete with grime, to add emphasis to the point. "The experts plan to place a huge ring of *cement* underground around the foundations" (6). The confusion of *cement* with *concrete* is quite common.

Other examples can be noted. "This awful structure must surely be the embodiment in *concrete* of the attitude which spawned…". "If the planners persist in putting *concrete* and cars before people, the city will die." "Dublin is depressing enough in the grey weather, with its lines of *concrete-rendered* housing in bleak and treeless streets." ".....a campus landmark to atone for the *concrete* cock-ups of the sixties and seventies." ".....an orgy of *concrete*.......". ".....a huge steel-and-contrete butterfly....." "A jungle of *concrete,* glass and indifference,.......tall inanimate staring columns of *concrete* blocks......"

The point at issue here is not the validity of what is being stated but rather the use of one of our best known building materials to create a distasteful picture by association.

Why is this so? Why do people misuse language like this? Lack of education? A wrong type of education? A distrust of what engineers, architects, planners, etc. do, especially if it is out of taxpayers' money, in order to create a civilised living for all? Society needs houses, clean water, comfort, buildings of every type, good roads and railways and so on. Whether tastefully done or not, such monuments to society's needs must not, in the eyes of the beholders, be a blur on the environment. The deliberate choice of the word *concrete,* noun or adjective, to describe any of them, whether the concrete is visible or not in the finished product, immediately condemns them to the realm of awfulness.

The concrete industry has adverted to this phenomenon on occasions in the past. In 1991, that marvellous magazine *Concrete Quarterly,* now sadly no longer with us (although at the time of writing, hopes of its revival exist), Martin Clarke quoted Max Hutchinson as saying that "The words concrete and architect occupy a special place in the public's affections. It is a secure place rather like a prison wing reserved for special offenders" (7). However, he also refers to another, and more positive, side to concrete. Political commentaries and speeches are liberally, and increasingly, dotted with "concrete proposals" and "relationships being cemented" with "concrete solutions" and plans for "concrete action". There is demonstrated here a certain dichotomy, a trace of ambivalence about the term *concrete*. While being, on the one hand, regarded as synonymous with ugliness and lowliness, it is also, on the other hand, used to convey an impression of strength, solidity, reliability and durability. But not beauty.

A similar contradiction can be seen to exist in people's attitudes to animals. Pets are generally loved for lots of good reasons, their beauty, companionship, attractiveness, cuddliness etc. Yet, at the same time, the English language is full of the most derogatory references to animals. For example, sly people are compared to foxes, horrible-looking foods are called dogs' dinners, a person may be as blind as a bat or as dirty as a rat. There are many others. This of course may be of little comfort to lovers or promoters of concrete. But it is nonetheless an interesting aside to the thinking of unthinking people.

Why the Dislike of Concrete?

Is this popular and largely negative view of concrete a recent phenomenon? Probably not. According to John O'Reilly, a Dublin architect, in his inaugural address to the students of the Advanced Concrete Technology course in Dublin recently (8), there was a certain "esprit nouveau" abroad in the early days of the 20th century. It was the beginning of the "modern movement" in architecture and the advent of the "machine age". A new spirituality came into being, of which, according to Van Doesburg, the machine was the creator. There were indeed machines for all sorts of occasions—physical as well as spiritual. A book was described as a "machine for reading" (Paul Valéry), or "a machine to think with" (I.A.Richards). A bridge was "a machine for taking one across". A painting was "a machine for moving us" (Saugnier), the theatre "a machine for acting" (Eisenitein). All very dignified and not very controversial. However, when Le Corbusier described a house as "a machine for living in", this was regarded, understandably, with outrage. Houses, and the homes that they enshrined, were now being called machines. The idea of one's own cosy friendly intimate private sanctum being referred to as a "machine" was just too much. In particular the flats and apartments, several made of reinforced concrete with flat roofs, which were homes to many families, were now equated in the public eye with "machines" and the main material of which they were made, concrete, was branded as not merely anti-home but also as "Bolshy".

The assault on concrete had begun and over time it led to the brainwashing of the popular imagination in regard to the fairly new material—reinforced concrete.

A further ingredient in the depopularisation of concrete was the use of the innocent and descriptive architectural terms *brutal brutalist* and *new brutalism* to describe some of the exciting concrete architecture of Corbusier *(beton brut)* The terms, unfortunately if understandably in the English-speaking world, misfired and *brutal* was adopted as a word to express all the public found crude and rough and awful with concrete.

Who likes concrete?

Such is the title of this paper. The only satisfactory way to answer the question is to ask a representative sample of the population whether they like concrete or not and to record the results.

A small survey was undertaken in 1995 as part of a larger project by a pair of final year civil engineering students in University College Dublin (9).

Among the questions asked was *"What do you think of concrete?"*. 68% replied that it was either boring, horrible or dull, 23% said it was nice and 9% said more could be done with it. To the question *"Could you name a concrete building?"*, 62% answered yes. However, two thirds of this number, when asked to name a concrete building, named one that was not concrete!

Regarding the colour of concrete, 85% of the sample got the colour correct (grey), the remainder describing it as either white, yellow-white or green-grey. None of the sample, when asked, was aware of the fact that pigments could be used to colour concrete. The texture of concrete was variously described as hard (23%), rough (23%), smooth (14%), cold (11%), like stone (11%), solid mass (9%), patterned (6%), plain (3%). This is probably as true a reflection of concrete's variety as one could desire. While these figures reveal an apparently objective view of concrete surfaces, nevertheless, in view of other answers, one suspects that an underlying emotional response lay behind the choice of terms used, e.g. hard, rough, cold.

The survey was carried out on several of Dublin's busy streets and a total of 117 people were asked a series of questions. Perhaps a more widespread and more representative survey of people's attitudes to concrete could be undertaken at some higher level. For example, an organisation such as one of the Concrete Societies of these islands, or the BCA, the Institute of Concrete Technology, the Irish Concrete Federation, etc. could undertake such a survey. The various concrete research centres and departments of civil engineering in the Universities and Colleges of Technology of these islands could also be enlisted to help. The concrete industry would stand to gain from the results of such a survey.

The Perfect Concrete Finish

Architects are the undoubted masters when it comes to shape and surfaces. They are the experts in transforming a given volume of space into a three-dimensional edifice that will fulfil all that the client has asked for—and possibly much more—at a cost that is also not without the client's agreement. Until a building is built and occupies real space, a drawing is the main representation that will convey the shape of the architect's proposals. However, it will not, it cannot, convey any appreciation of the surface appearances, colours, textures, shadow effects, subtleties of light and shade, etc., except in a very crude way. Surfaces can be specified, described, shown on samples. But such methods are at best only a good guide, but are no guarantee of the success of the intended effect. At worst, they may bear little relation to realised surfaces. Models or sample panels, are an improvement on the drawing, but are expensive and time consuming and are only appropriate for special buildings or to illustrate complex relationships.

Concrete is a particularly difficult medium when it comes to its appearance. In the first place, as already stated, the public has a rather jaundiced view of the material. Secondly, concrete is a composite material that is manufactured largely from the rather raw rude almost random little tamed materials of nature. There are many stages between the architect's conception of a concrete surface and what the client eventually sees for the first time in the flesh. The architect will intimate what he or she wants. The consulting engineer will draw up a specification to which the surface should conform and possibly also specify how it should be achieved. The ready mix supplier will manufacture the concrete, the contractor and all his men of varying skills or none will place it and finish it. And the client will live with it thereafter.

But in between these major stages of construction, the potential exists for many things to go wrong. The person delegated to write the specification may not have a full appreciation of the many factors and their interaction that can affect the final appearance of a mass of concrete. After the ready mix supplier has designed the mix and manufactured the concrete, it then has to be loaded into a truck, transported to site along congested streets, roads, lanes, in whatever weather conditions obtain on the day and all within a given time. On site, the concrete must be transferred from the truck to the formwork—directly or via a dumper, conveyor, crane or pipeline.

Parallel to all this activity, but eventually converging with the concreting, the required reinforcement will have been determined, detailed, scheduled, ordered, cut, bent, bundled, labelled, transported, assembled and so positioned in the formwork that it will simultaneously fulfil its structural duty and also be adequately protected from corrosion for the rest of its life. Even here, the potential exists for something somewhere along the line to go wrong in a way that could affect the long-term appearance of the concrete. The concrete, containing its cargo of reinforcement, must next be adequately but not too enthusiastically compacted by vibration. The next few days will be crucial—assurance of adequate curing is vital. Freezing weather, hot days, windy weather, variable humidity, all impact in their own way on the curing period. There are other matters that add to the drama. The materials and construction of the formwork play a large role in how the finished concrete will look. The size and extent of the pour may cause headaches—human and concrete—if either is very large.

And then, when the formwork is stripped, the conception period is over and the concrete is born—life now begins in earnest for the concrete. The rain will lash, the wind will howl, the sun will scorch, the frost will bite, dust will blow and settle and unsettle again, oxygen and carbon dioxide will both permeate through the concrete, each with its own hidden agenda over time. The pollution of a billion exhaust/kilometres will seek refuge in the pores of the concrete. The spores of algae and fungi and mosses will conspire to see if a living can be made off the concrete surface. Life with the neighbours, the bricks, stones, windows, polymers, metals, etc., may be a little strained. On top of all these there are the effects of the stresses and deflections and cracking due to the variety and extent of dead and applied loads. In brief, weathering has begun!

It may seem, after all this, that to achieve a concrete finish that will be perfect for all time is impossible. It isn't. But it is so nearly impossible, even with present day technology, that it would be unwise to try always to get it. Perfection can only be bought at a price and the price is not always acceptable or available.

This is a simple fact of life. But it does not help the perception of concrete by the general public. Whatever about perfect concrete, will people accept imperfect concrete? There is a challenge here for the whole concrete industry, namely, to educate the public to understand and appreciate concrete at both a conscious level and more importantly if less easily, at a subconscious level. The public may still not like it, but it could now rationalise its dislike.

A LIMIT STATE OF APPEARANCE

Limit States

Modern structural design codes allow for two principle limit states. The first and more important one is the limit state of collapse, or more commonly, the ultimate limit state (ULS). The primary concern of the ultimate limit state is with safety of people and is not an issue as far as this article is concerned. Secondly and less importantly, there is the serviceability limit state (SLS). According to IS326 (10) and BS8110 (11), structures should *"not become unfit for their purpose by collapse, overturning, buckling (ultimate limit state), deformation, cracking, vibration, etc. (serviceability Limit states) and that the* *structure should not deteriorate unduly under the action of the environment over the design life, i.e. will be durable."* (Clause 2.2.1).

What, however, is not included, under the serviceability limit state is a state that would reflect an observer's acceptance of, or satisfaction with, the appearance of a building, whether it be the apparent structural safety of the building or the purely visual aspects. Part of what would impinge on the casual passer-by, even if only subconsciously, would be the visible surfaces. And if concrete is a part of the building and is exposed to view, then, taking account of its particular relation with adjoining surfaces, it must be satisfactory to the casual observer.

A Limit State of Appearance

Since a serviceability limit state of public acceptance could not be easily specified by code, a case can be made instead for a serviceability limit state of appearance. In the same way that cracking and deflection are controlled by SLS requirements, then similarly, a serviceability limit state of appearance could be formulated by code. It might of course conflict with other limit states or durability demands but this is not a problem—the more demanding criterion would in all cases be the determining one.

An observer's impression or perception of a building surface, at a conscious or subconscious level, depends both on the state of the surface and the distance of the observer from the surface. Another factor can also be included here, namely, the importance, or status, of the surface in the building. These three parameters—importance, distance and surface quality—could be broadly related by means of a "perception chart". See Figure 1.

A scale of five would be appropriate for each parameter, as follows:

1. Importance:

   Grade A—very important, e.g. the nature and function of the building demand that the concrete surfaces must be of the very best quality;

   Grades B, C, D—intermediate between A and E;

   Grade E—not important, e.g. in some part of a building, the quality of the finish may be deemed not to matter at all visually.

2. Distance:

   Grade 1—Adjacent and tactile and/or easily seen, e.g. stairs, walls, columns, footpaths;

   Grade 2—near and visible but not touchable, e.g. ceilings, high walls, bridge parapets, basements;

   Grade 3—visible in the near distance, e.g. factories, large buildings, small chimneys, bridge structures;

   Grade 4—visible in the far distance, e.g. high chimneys, power stations cement works, bridges;

   Grade 5—remote or not visible to the casual observer, e.g. foundations, dams, offshore structures, hidden wall surfaces.

3. Quality of surface finish. A scale of five would encompass a quantitative or a qualitative

description for each of several surface conditions:

Grade I—the very best quality;

Grades II, III, IV—intermediate between I and V;

Grade V—the lowest quality.

The various surface conditions, each with its own scale of five, would cover at least the following:

i. cracking—from invisible at close quarters(or none) to visible at a distance;
ii. tactile—from very smooth to very rough (terms such *as off the shutter, fairface*, etc., are too vague). This could possibly be subdivided to deal with exposed aggregate finish, bush hammered finish, board marked finish, etc., as well as applied cementitious finishes, each with an associated scale of five;
iii. colour variation—from none to, say, noticeable at a distance of *x* metres at *y* months;
iv. joints—from specially treated for invisibility or acceptable appearance to no particular control;
v. honeycombing—best described quantitatively, e.g. 1% to 10% or more of a surface area. The randomness of its occurrence should deserve attention also;
vi. leaching—this could be broadened to include efflorescence, algal bloom, fungal growth, moss, lime leaching, and a quantitative scale would possibly be appropriate. There are difficulties with this since none of these phenomena are obvious on striking of the formwork and may take years to become evident;
vii. shutter marks—similar approach to joints.

Thus, a Grade III surface finish would consist of a grade 3 in each sub-element in the subset of finish descriptions. Therefore, rather than having a proliferation of possible combinations of finishes, there would be only five grades, each grade consisting of one subset from the list of seven finish requirements.

To operate the system in practice, a designer would need to describe the details of the seven (or more) finish descriptions and agree them with, or present them to, a client. Then, a judgement would be made on the importance of the various concrete surfaces (A to E) and a judgement made on the distances of the surfaces from some agreed observation point (1 to 5). These two, when plotted on the “perception chart” of Figure 1, would give the surface quality grade required. For example, a C-rating for importance with a 1-rating for distance would require a Finish Type III. Similarly, a building of top rate importance, i.e. with an A-rating on the importance scale, but only visible in the far distance, would have grade IV surface finishes, externally anyway.

CONCLUSION

The question “Who likes concrete” has not really been answered. But if careful attention is paid to concrete finishes from the point of view of the casual observer as well as for durability purposes, perhaps along the lines indicated, then maybe people, over time, will come to accept that concrete is not really so bad after all. Indeed they might even agree than concrete can be beautiful. And some other word will then have to be used to conjure up the nastiness and distaste that seems to be the present purpose in the public eye of the word *concrete*.

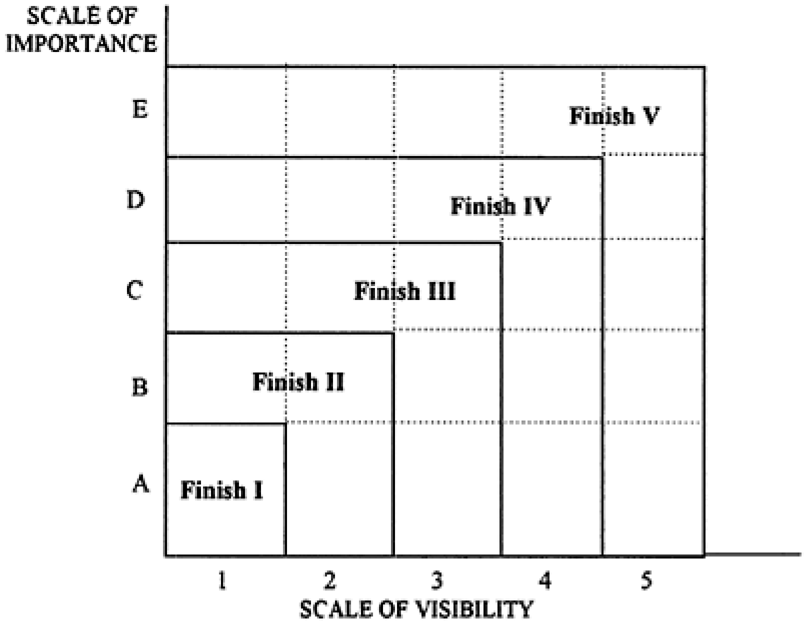

Figure 1. A perception chart

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PERFOMANCE CRITERIA: THE PAINS AND STRAINS OF CHANGE

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ABSTRACT. Since 1989, Romania has endeavoured to adjust to the needs and demands of a market economy. Technologically, in the construction field, the expertise of Romanian engineers is of a high standing and numerous sophisticated theoretical concepts have been developed over recent years. However, it is the translation of such concepts into practice that has needed considerable effort and changes to create systems and procedures appropriate for Romania in the 1990's.

One of the most pressing needs has been to modify the legislation under which the design and construction of buildings are built. In addition to this has been the crucial process of ensuring that the performance concept and the need for producing sensible criteria has been developed and effectively implemented. In particular, the identification and precise determination of client needs by the designer/constructor, in both qualitative and quantitative terms, has been a demanding part of the process.

The paper will address the means by which these 'new' concepts have been adapted and adopted in Romania. It will highlight the problems that have been experienced and expound upon how they are being resolved in an emerging free market economy.

Keywords: Construction, performance concept, performance criteria, legal liabilities, cultural differences.

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INTRODUCTION

The design and construction process has as its most important goal to identify the needs of the client and to reconcile them with the design, technological, economic and architectural aspects of the project. These aspects have to answer, in a suitable way, to the ecological and social requirements. This is a difficult problem to solve if the financial resources of the client and society generally are constrained, which is often the case for those countries in transition to a market economy.

In the past, Romanian civil, structural, building and services engineers developed a high standard of expertise. In structural design, especially of reinforced, prestressed or partially prestressed concrete structures and of steel structures, this was particularly the case. For these design engineers, all their efforts were concentrated in obtaining strength, stability and durability of the structure by complying with the main building material (cement, steel, timber) consumption indices fixed by arbitrary regulations ordered by the communist party. The party's 'wish' was to constantly decrease the indices in order to increase 'efficiency' in the building process. On occasions, because of the very strict liabilities, some of the structures, especially in areas affected by earthquakes, were over-designed in order to protect the designers and builders who were responsible for the strength and stability of the building for its entire lifetime.

Little or no concern was given to the finishes or to the comfort of the buildings and although the architects tried to influence the aesthetics of buildings (mainly in blocks of flats), it was very difficult to achieve this. This was because of the high level of standardisation of these buildings since the majority were constructed of large prefabricated reinforced concrete elements.

THE PERFORMANCE CONCEPT AND THE PERFORMANCE CRITERIA

Specialists in the construction field increasingly became dissatisfied with the way these blocks of flats looked and particularly with the level of comfort offered to their tenants. Therefore, these specialists brought to the attention of the authorities the need to improve the quality of housing, which would also improve the health of the occupants.

It was with the help of the performance concept, which was introduced in the different regulations concerning the quality of the buildings, that these specialists tried to change the entire philosophy of the house building process. Before the introduction of the performance concept, the focus was on the quantity of buildings produced in order to satisfy the social demands prescribed by the socialist and communist economy. The effect of this approach was that quality was neglected. Through the introduction of the performance concept, it became possible to address quality issues and standards were subsequently able to be improved.

There was and still is a difficult problem to solve in Romania because of the small range and inadequate supply of building materials available for finishes and because of the necessity to create a culture for quality. Previously, people typically accepted the flat offered by the state at a low rent without complaining about its quality or comfort. The designer and the builder were concerned about the strength and stability aspects of quality and did not consider the quality of finishes as important. However, this was in great contrast to the high specification finishes of some residential buildings built for some communist party members compared to the quality offered to the public generally. The House of the Republique in Bucharest with its exceptionally high quality is only one example of the high level of craftsmanship and skill of Romanian builders.

The performance concept is a methodology which enables the establishment of the necessary quality characteristics of a building to be met. In this way, the user requirements can be identified without necessarily taking into account the means by which they will be achieved. It will, therefore, be appreciated that the introduction of the performance concept in Romanian building regulations was so important. In addition to the criteria of strength and stability, fire risks, impermeability and durability, it is necessary to add criteria which focus on physical and psychological comfort', including hydrothermal, atmospheric environment, acoustics, artificial lighting, tactile, hygiene, anthropometries, radiation and sun brightness, electric field and ionic density, security, privacy and servicing continuity. Each of these help to ensure that building occupants are provided with good living accommodation. It should be appreciated, too, that some of these requirements existed either explicitly or implicitly in the former regulations in Romania, but they could only be requested rather than demanded. Many others have been identified by the author of this paper as a result of a deep study of human needs and requirements being undertaken as part of doctoral research. It is not the intention to insist on the functional architectural requirements which are increasing the quality of buildings. However, the freedon to create without being forced to comply to the strict rules of standardisation brought a significant change in project originality with respect to site integration, general space organisation, flexibility and aesthetics. Further, to create a culture for quality means much more than a quality flat or house. It has to take into account the urban integration, the quality of the services offered and the built environment as a whole. People, too, have to be taught to be more demanding when considering the environment in which they live and should ask for the provision of schools, leisure facilities and shopping areas which will improve the quality of their living environment.

The distance from an individual's house to their place of work is also an issue viewed differently by people from different countries. The Anglo-Saxon approach generally considers that to travel up to 25 miles from home to their place of work is reasonable and therefore presents few problems. In the Latin culture and the culture of some Slavonic countries in the East or Central Europe, it is essential to have a house as near as possible to the place of employment. This need is not only a matter of culture, but also a matter of the economic level of development. In countries with poor infrastructure, poor local travel systems and where access to and use of a car is considered a luxury, it is difficult to consider a house if it is a long way from the place of work.

It is clear that the level of these criteria and not simply their quantity is dependent upon the wealth of the country. For example, the minimum temperature in wintertime which should be provided in a house in France, as with most European developed countries where a temperate climate exists, is 24° Celsius. In Romania before 1990, the figure was 18° Celsius (but this was never respected). Now, this has been raised to 21° Celsius. In 1995, this new figure is still not being achieved. In reality, such requirements can only be recommendations because on several days per year, these figures cannot always be achieved even in Western European countries. In Romania, to even achieve figures of 21° Celsius for the majority of the time is still a major goal.

Another perspective on building is what is considered to be their lifespan. Should the philosophy be that buildings are considered as a 'consumption product' with a lifespan of 30 years maximum for houses as in America, or should the lifespan be 100 years as was the case with the blocks of flats in Romania. In the 30 year lifespan example, timber frames and lightweight materials are used, whereas in the 100 year lifespan example, reinforced concrete elements provide the framing and cladding system. Further, the 30 year lifespan philosophy does not provide a means of transferring the culture and tradition onto the next generation. One therefore builds for one generation mainly in the fashion of the day using the latest materials and services. In the 100 year lifespan, a means of transferring to the next generation a heritage is provided. All the processes from inception to completion are different and the differences are not only in technology and philosophy, but also in culture. Comparisons between Northern and Southern Europe are difficult, not least because of climatic differences. Likewise, it is not sensible to compare houses built in Japan with those built in Africa. This is not only because of climate but also, and importantly, in the concept of family life which is radically different. Finally, the potential for natural phenomena occurring, such as earthquakes, will influence the design, technology and materials used in buildings.

CONTRACT MANAGEMENT, PERFORMANCE CONCEPT AND LEGAL LIABILITIES

The improvement of quality by Romanian specialists and the crucial role played by the performance concept and the performance criteria are major inputs. These provide an open feedback system which is the quality assurance system (see Figure 1). During the contract execution, contractors have major responsibilities from the point of view of quality. Romanian Law No. 10/1995 "Law Concerning the Quality in the Construction Industry" [1] sets out the requirements of the building works, the quality system and the obligations and responsibilities of all the actors' taking part in the erection of a building. In order to achieve a high quality building, the law demands the following requirements, which shall be met during the lifetime of the building. These are: strength and stability, safety in operation, fire safety, hygiene, minimum health standards, environmental recovery and protection, thermal and weather protection, energy saving and noise protection.

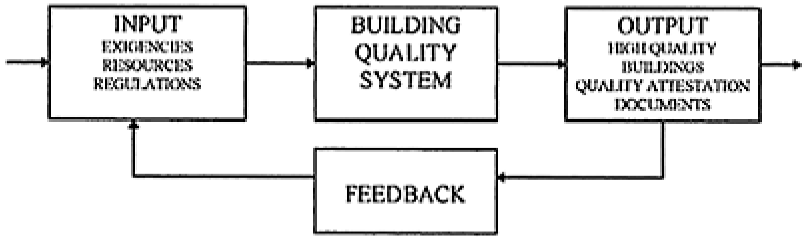

Figure 1 Building Quality System—Total Quality Management

In accordance with this law, the contractor, in summary, has the following primary responsibilities:

a) to inform the investors on the lack of conformity or agreement in the project with the aim of solving such problems;

b) to begin works only with approval in accordance with the present legal requirements and with projects checked by chartered project experts;
c) to ensure the proper quality level through his/her own quality system, devised and achieved by his/her own staff, with the execution and technical responsibilities being given to chartered personnel;
d) to identify the factors involved in the checking of the works in each building phase and to ensure the requirements are achieved;
e) to give solutions for lacks of conformity, defects and dysfunctions during erection only on the basis of the designer's solutions and with the agreement of the investor;
f) to use only products and procedures provided in the project. These products and procedures have to be checked to ensure compliance with the quality standards. The replacement of the products and procedures in the project by others with similar features is only possible if the designer's solutions and investor's agreement are obtained;
g) to carry out all details in order to meet the quality requirements;
h) to inform the "State Control Office for Buildings, Public Works, Urban and Land Planning" within 24 hours if technical disturbances appear during erection;
i) to submit for inspection only high quality works meeting the requirements and which have the necessary technical documents needed to fill in the estate documents;
j) to carry out all measures established by control or checking bodies or by work reception documents;
k) to repair at the contractor's own expense any quality failure either caused during erection or during the guarantee period;
l) to bring back the temporarily engaged land to the original state when works are completed;
m) to decide the exact responsibilities of all participants taking part in the process of production, ie responsible parties, co-workers, suppliers and subcontractors. These responsibilities will be assigned according to the performer's own quality system and legal provisions.

The legal penalties for non-compliance with the law are very severe. For a loss of strength and stability of a building which directly leads to loss of life or serious injury or partial or total destruction of the building or other severe consequence, could lead to imprisonment of between 5 and 20 years. Where other non-compliance occurs, fines ranging from 10,000 to 30,000,000 lei (4300 lei=£one sterling in December 1995) are applied.

The penalties described above might be regarded as harsh, but they are necessarily so to ensure that total quality management systems are applied to the entire building process. The quality assurance system in Romania is not yet fully in place, but it is essential for the building to be designed to high standards irrespective of this.

The law also prescribes the obligations of the owner or the tenants of the building. This is a very sensitive issue because although the owners and tenants are forced by law to maintain their buildings to a high standard, they often cannot comply with these requirements because of the lack of financial resources. This is also the case with stateowned buildings. It is, therefore, necessary to create, as an essential step in the insurance system, a facility to enable owners to receive low-interest rate loans to enable maintenance work to be undertaken. This will also ensure that the previous investment in the built environment will be protected.

CONCLUSIONS

This paper describes the difficulties that a country such as Romania in transition to a free market economy faces when it endeavours to achieve high quality buildings in an effort to satisfy the needs of users. The change is not only difficult because of economic, technological and technical constraints, but also because of the necessity to implement a new legal framework. The creation of social and public services to produce the necessary infrastructure is essential too. But what is arguably even more difficult is to create in people a 'quality culture' and for them to realise that the only reason for such systems is to help satisfy their requirements. Implicit in all the issues on quality is the need to realise the impact that construction has on the environment and be aware that the concept of environmental protection and sustainability is of paramount importance. Only when all these issues are fully understood and implemented will the pains and strains of change diminish.

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APPLICATION OF A STATISTICAL METHODOLOGY FOR THE EVALUATION OF THE POTENTIAL MARKET OF NEW CONCRETE PRODUCTS ON THE CONSTRUCTION INDUSTRY

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ABSTRACT. This paper presents the results of a case study carried out for the evaluation of the impact of new concrete products or technologies on the construction market. The applied methodology consisted on the planning and implementation of a survey of the Chilean construction industry and its later statistical analysis. Stated Preference data and the discrete choice Logit models were used to determine the potential demand function for the product. The model allows to know the probability of choosing a new product compared with a traditional one as a function of certain product attributes. It is worth recalling that these tools are frequently used in market research and transportation engineering.

Keywords: New, Innovative, Concrete, Products, Potential market, Construction. Industry, Evaluation, Statistical, Stated- preference, Discrete Choice, Logit Model.

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INTRODUCTION

The present work forms part of the study "Assessment of the Potential Market for Minimum Cracking Concrete, MCC" [1], performed by the authors for a Chilean cement company. This paper examines a methodology to evaluate the impact of new concrete products on the construction market and allows to know the probability of choosing a new product compared with a traditional one as a function of the "level of service" of certain product attributes. The procedure was used to quantify the potential users market of fibre reinforced concrete (FC). The characteristics and the average willingness to pay of the Chilean concrete market for the new product were studied. The authors express their acknowledgement for the technical and economical support given by the company POLPAICO S.A. and to Andrés Iacobelli for his unselfish technical assistance.

METHODOLOGY OF ANALYSIS

The applied methodology consisted on the planning and implementation of a survey of the Chilean construction industry and its later statistical analysis. The survey was carried out by sending two questionnaires to consulting engineers and constructors. The aim of the first part of the questionnaire was to identify and quantify the main problems encountered in concrete construction and their causes, and to evaluate the required concrete properties on relative terms depending on its use. The objective of the second part was to determine the potential demand function for the product as a function of the level of service of certain technical attributes and the willingness to pay of users (concrete consumers) for those attributes. The procedure used to analyse the data obtained from the first part of the questionnaire consisted on summarizing the results in tables and figures as a percentage of the number of responses to specific options on each question. Designers and constructors answers were treated separately.

The stated-preference/intentions technique (choice type) [2] was used for the design of the second part of the survey. Each person was faced with nine hypothetical cases. In each of them the respondent was asked to choose between the use of traditional concrete (TC) or fibre reinforced concrete (FC) for a specific application (pavements, hydraulic structures or shotcrete mortars). In each case the level of service offered by the FC attributes, with respect to those offered by TC, were presented to the respondent. Those attributes were: price, reduction of cracking (%), increase of abrasion resistance (%) and increase of impact strength (%). To model the users behaviour the random utility theory was used [2]; thus assumes that individuals select an option from a finite set of alternatives based on the relative attractiveness of the option, i.e. they always select that option which maximises their net personal utility. However, a wide range of variables or attributes can define the utility that a particular product can offer. Also, not all individuals consider the same variables for their choices. Therefore, the net utility of a individual is modelled as the sum of a representative utility (or utility function from now on, which is a function of attributes that are assumed to be considered by all individuals) and a random component (which consider that the modeller does not possess complete information about all elements considered by individuals making a choice, together with any observational errors). The form of the utility function is linear, e.g. for each alternative of this particular case it is as follows [2]:

$$V_q^{TC} = \theta_{q\,\mathrm{Pr}}^{TC}\,\mathrm{Pr} + \theta_c \tag{1}$$

$$V_q^{FC} = \theta_{q\,\mathrm{Pr}}^{FC}\,\mathrm{Pr} + \theta_{qCr}^{FC} C_r + \theta_{qRa}^{FC} A_r + \theta_{qIs}^{FC} I_s \tag{2}$$

where, V_q^{FC}, V_q^{TC} Utility function of individuals belonging to segment q for concrete with (FC) and without (TC) fibres, respectively.

$\theta_{qi}^{FC}, \theta_{qi}^{TC}$ Parameters for the i[th] variable in the utility functions of segment q, for the FC and TC alternatives. They represent the relative influence of each attribute in terms of contribution to the overall satisfaction produced by the alternative (marginal utility).

i : Variable representing and attribute. The attributes i were the following: Pr=Price (UF/m^3), UF is a commercial interchange unit used in Chile and its value is approximately US$ 32; C_r=Reduction of cracking (%); A_r=Abrasion resistance (%); I_S=Impact strength (%).

θ_c: Calibration or modal constant.

Because the net utility of an individual is a random variable it is only possible to work with probabilities, i.e. to predict the probability of choosing a certain alternative, according to the level of service offered by each of the available options. From the utility functions of each group q and defining certain levels of service for each option, it is possible to determine the probability of choosing each alternative. Because this is a binary choice the sum of the probabilities is equal to:

$$\mathrm{P}_q^{FC} + \mathrm{P}_q^{TC} = 1 \tag{3}$$

where, $\mathrm{P}_q^{FC}, \mathrm{P}_q^{TC}$: are the probabilities of choosing FC or TC, respectively.

The Logit model was used to model these alternatives; it is capable of determining the probability of selecting a particular alternative, only knowing the function defining the representative part of the net utility [2]. For this reason the main efforts of the questionnaire were focused on the definition of the representative utility functions of the alternatives with and without fibres.

DISCUSSION OF RESULTS

Market Characteristics

Concerning the market characteristics only the results related to the main problems encountered in concrete construction and their causes will be deal with, because lack of space. More information can be found in reference [1]. Figure 1 shows the most frequent problems encountered in Chilean concrete construction works. An interesting aspect highlighted on this figure, is that concrete cracking is the most significant problem from the point of view of both designers and constructors, representing around 40% and 33% of the answers respectively; thermal cracking appears as the most frequent problem for constructors (25%) and designers (19%). This result also indicates that the most severe problems are encountered with cracks occurring during the first month of the concrete life. Also, results showed that construction problems appears to be by far the major cause of failure (41%).

Market Willingness to Pay

The purpose of the second part of the survey was to determine the utility functions of each alternative, i.e. fibre reinforced concrete (FC) and common concrete (TC). As already mentioned, each respondent was faced with nine different cases. Each of those was considered as an independent observation; therefore, our sample of 29 questionnaires was equivalent to 261 observations or *pseudo-individuals*. The design of the questionnaire allows to verify apparent irrationalities in the answers; 13 pseudoindividuals were discarded after the revision process. Therefore, the final sample was 248 observations. These were divided according to the professional activity of the respondent (model 1: designers or model 2: constructors) and the particular field of application considered (model 3: hydraulic structures or model 4: pavement construction). The answers related to shotcrete mortar (less than 20 observations) were discarded because of the lack of statistical significance of the reduced sample. Table 1 presents the results for each model [1], where the number in brackets represents the statistical significance level of the variable (t-ratio); an absolute value higher than 1.96 means that the parameter for that variable is significantly different from zero, at 95% of confidence level.

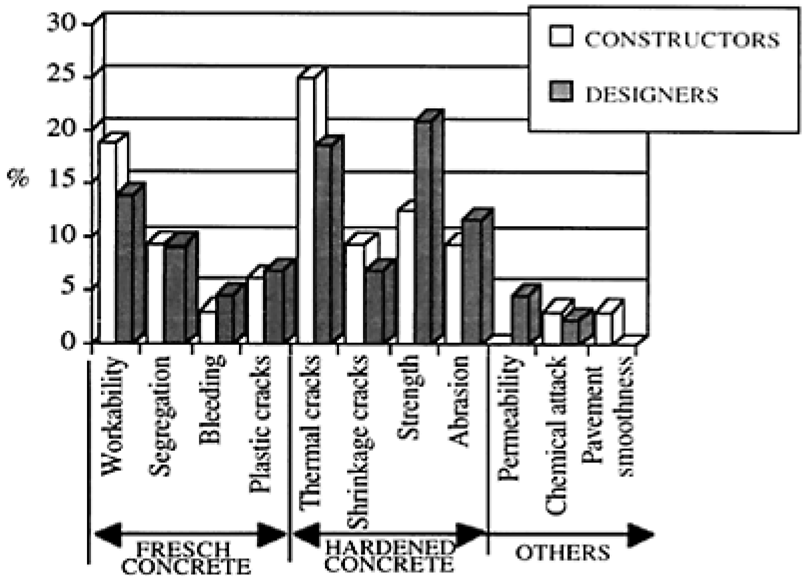

Figure 1 Most frequent problems in Chilean concrete construction works

Table 1 Summary of model parameters and willingness to pay for each segment

Model for segment q	N°	Model parameters (t-ratios)					Willingness to pay	
		Price	Cracks	Abrasion	Impact	constant		
		θP_r	θ_{cr}	θ_{Ar}	θ_{Is}	θ_c	WP_{Cr}	WP_{Ar}
1	158	−4.556 (−4.9)	0 .03604 (3.9)	0.03452 (1.6)	0.004863 (0.6)	0.1502 (0.2)	0.25	–
2	90	−3.594 (−3.2)	0 .02332 (2.2)	0.02460 (0.9)	0.0116 (1.1)	0.5448 (0.6)	0.21	–
3	122	−4.807 (−4.1)	0 .03558 (3.2)	0.06815 (2.6)	0.01101 (1.2)	0.5025 (0.6)	0.22	0.45
4	110	−4.799 (−4.3)	0.03493 (3.3)	–	–	0.06391 (0.1)	0.23	–

The subjective marginal value or willingness to pay of a group for the improved characteristics offered by FC concrete can be estimated for the three technical variables considered in the model. However, in the analysis that follows, only the value of those variables with statistical significance was determined as follows (i represents a technical characteristic, e.g. Willingness to pay of Cracks WP_{Cr}):

$$WP_i(US\$/m^3/\%) = \left[\theta_{qi}(1/\%) \Big/ \theta_{qPr}(1/UF/m^3) \right] * 32(US\$/UF) \qquad (4)$$

The WP is the extra amount of money in US$/m^3, that a individual should be willing to pay for each 1% improvement of a specific characteristic offered by a new product [2].

The most significant variables for the group of designers are clearly price and percentage of crack reduction [1]. The willingness to pay of designers for FC concrete offering a 1% reduction of cracking (VMCr) is 0.25 US$/m^3/1% crack reduction. Therefore, if FC were capable of reducing by 50% the amount of crack formed, the designers would be willing to spend up to US$ 12.5 per cubic meter in FC more than the amount they would spend if they bought a traditional product [1].

Similarly, the statistically significant variables for the group of constructors are price and percentage of crack reduction. For this group of professionals WP_{Cr} is 0.21 US$/m^3/1% crack reduction. It is interesting to note that the designers' willingness to pay for reduction of cracking is 20% higher than that revealed by the constructors [1].

In the case of hydraulic works not only price and percentage of crack reduction are statistically significant; also abrasion resistance is important. The willingness to pay of abrasion (WP_{Ar}) is 0.45 US$/m^3/1% abrasion resistance increase; in this model WP_{Cr} is 0.22 US$/m^3/1% crack reduction. The results show clearly that individuals working on hydraulic structures values much more the abrasion resistance than the reduction of cracking (WP_{Ar} is more than 100% higher than WP_{Cr}) [1].

For the group of pavement construction, the abrasion resistance and impact strength variables were not included because they had significance levels far below 1.96. For this reason the model calibration was performed only for price and percentage of reduction of cracking. The calculated WP_{Cr} value was 0.23 US$/m^3/1% crack reduction [1].

Segmenting the Market

The determination of the market segmentation requires to know the probabilities of choosing each one of the alternatives (type of concrete), calculated from their utility functions (V). The expression to calculate the probability of selecting a FC concrete with certain characteristics, i.e. with a given price and specified technical properties for crack reduction, increased abrasion resistance and impact strength, is as follows [2]:

$$P_{FC} = e^{V_{FC}} \Big/ \left(e^{V_{FC}} + e^{V_{TC}} \right) \qquad (5)$$

For example, if it is desired to know the proportion of the hydraulic structures market selecting FC the expressions for V in model 3 must be replaced in the above formula. Certain values must be assumed for the price of each option as well as for the percentage of technical improvements of FC concrete compared with TC concrete. The following limits were used: 30% and 90% crack reduction; 0% and 20% abrasion resistance increase; 10% and 60% impact strength increase. Table 2 shows an example of the results obtained when market segmentation procedure was applied [1].

Table 2 shows that it can be expected that more than 66% of the hydraulic structures market should use FC if it is defined as having 30% crack reduction, 10% impact strength increase and equal price and abrasion resistance than a traditional concrete. For the same FC (equal technical characteristics) but with a price 9.6 US$/m^3 higher than a TC concrete, then the percentage of the market interested in FC will diminished to almost 32%. The remainder part of the table can be analysed in the same way [1].

Table 2 Example of the market segmentation of hydraulic structures for FC product

Cracks reduction	Abrasion resistance	Impact strength	Price TC (UF/m^3)	Price FC (UF/m^3)	FC market participation (%)
30%	0%	10%	3.0	3.0	66.26
				3.3	31.71
				3.6	9.89
				3.9	2.25
60%	10%	35%	3.0	3.0	93.69
				3.3	77.85
				3.6	45.38
				3.9	16.42

CONCLUSIONS

This paper has shown the potentials of a new methodology for the evaluation of the impact of new concrete products on the construction market. One of the conclusions obtained is that Chilean concrete construction market considers as a recurrent problem the cracking of concrete at early-ages (first month); therefore, a product like fibre reinforced concrete should *a priori* occupy an important place in that market. With respect to the group division it can be concluded that designers have a much greater willingness to pay for crack reduction of concrete than constructors; this seems logical because the latter group perceives the effects of higher costs more closely than the former. An important difference can be appreciated between professionals working on pavement construction and hydraulic projects; in the first case they mainly value crack reduction, while the others give priority to improved abrasion resistance. Also, it can be said that the reduction of cracking is a characteristic valued for all individuals. Finally, it must be pointed out the very low value of the calibration constant (with no statistical significance); because this constant captures all those variables not considered in the modelling, it may be concluded that the chosen variables practically explain all the selection process.

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APPROPRIATE CONCRETE DESIGN

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ABSTRACT. The design and use of appropriate concrete technology to suit a particular circumstance is not an exact science. It requires sound engineering judgement and relies heavily on individuals' experience. In this paper, an attempt is made to highlight, through practical examples, some of the important issues concerning the use of concrete technology. It is emphasised that concrete technology means more than just the production of concrete of adequate strength and a wide range of issues have to be considered, e.g. the durability of the concrete element, its practicality and economy. On basis of the discussions, criteria are proposed to assess appropriateness in concrete design.

Keywords: concrete, strength, durability, practicality, economy, design, criteria for appropriateness.

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INTRODUCTION

There are two aspects to the design of concrete: materials and techniques. The purpose of design is to ensure that, under given circumstances, the most appropriate materials are selected and that compatible techniques of concreting are deployed so that the project specification is achieved.

There are broadly two types of specifications, either based on materials or on performance. With the former type of specification, materials requirements have already been researched and are laid down by the specifier, leaving little scope for modifications by the contractor. The “design” thus becomes the selection of the right techniques to go with the chosen materials. With the latter type of specification, the performance of the end product is specified and the contractor has the freedom in his design to suit the particular circumstance. This type of specification encourages innovative and economical design and is gaining popularity. Whichever type of specification is used, however, someone, somewhere has to translate the project specifications into specific requirements for concrete technology. Therefore, this paper takes a broad approach to concrete design and aims to highlight some important, and yet often neglected, considerations in the design process.

The design of concrete, unlike structural design, is not a well developed process. In the past, it was often taken as a means of achieving an adequate strength. Over recent decades, especially with the wide-spread use of reinforced concrete, it has gradually been accepted that concrete needs durability as much as strength, if not more so. This transition is justified by the fact that many structures have failed, not due to inadequate strength of their members, but their inability to stand up to deterioration caused by the environment. In today’s world, concrete design takes on new meaning and puts emphasis on the practicality, economy and environmental aspects of the design. As a volume construction material, concrete can be found in every developed and developing country and, with thoughtful design, concrete should and can meet the varying and sometimes very stringent requirements.

This paper aims to categorise and discuss some important issues concerning concrete design. It does not intend to become a design manual, but to provoke further thought by people involved in concrete design. On the basis of the discussions, criteria are proposed for the assessment of the appropriateness of a design.

CONCRETE TECHNOLOGY: DESIGN CONSIDERATIONS

It is considered that there are four major areas which concrete design has to take into account and they are strength, durability, practicality and economy. There are also some other factors, such as the environment, and they are discussed briefly under a separate heading.

While it is possible to design concrete with a range of properties, flexibility in design is often restricted by codes of practice and Building Regulations. It has been suggested that codifying aspects such as durability is not possible(1). However, amendments in codes and regulations are certainly required to incorporate ideas of modifying materials and techniques in order to achieve the flexibility in concrete design.

Strength

Concrete is usually classified by its strength, typically at an age of 28 days. Whilst this is convenient and does provide a base for comparison, it may not be the most appropriate under certain situations. For the design of strength one needs to know which of the following criteria to satisfy:

- strength at stripping shutters
- strength at different times of construction
- strength at application of full service load

The problem of accepting different strengths at different times is one of current code requirements. While early strength gain is often checked by testing cubes or cylinders at 1 and 3 days to determine whether formwork can be stripped or removed (as in slipforming), the full service load in many situations is only applied a substantial time after construction commences, hence a design strength at 90 or 180 days or sometimes even longer may be appropriate.

The significance of this change in specification is that it would increase the choice of cementitious materials and allows a more effective use of blended cements. It is known that, at the same water-cement ratio, the strength development of blended cement is slower than its ordinary Portland cement (OPC) counterpart, but its ultimate strength may be considerably higher[2,3], as illustrated schematically in Figure 1.

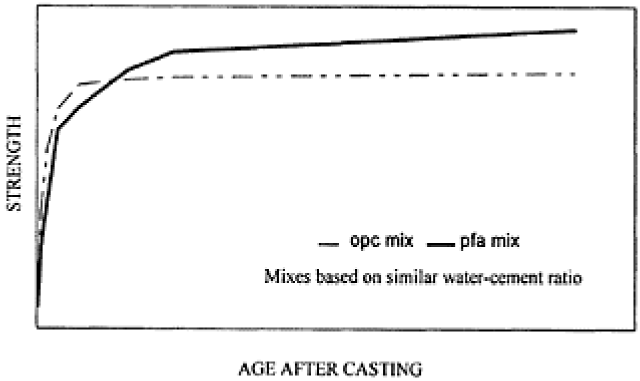

Figure 1 Schematic strength development

The benefits of using blended cement are:

i) It allows the effective utilisation of waste products, like pulverised fuel ash (pfa) or ground granulated blastfurnace slag (ggbfs), hence it is good for the environment.

ii) By using blended cements, concrete durability is often enhanced. For instance, the use of pfa as a cementitious component is recognised in current British Standards[4] as being beneficial in reducing sulphate attack in moderate sulphate exposure conditions. It helps to reduce the possibility of developing alkali-aggregate reaction[5] and to control, especially in the longer term, the ingress of chloride ions[6], which is often associated with the premature deterioration of marine structures.

iii) By allowing the use of blended cements, it helps to produce economic concrete mixes. For instance, if the 28 day strength requirement is replaced by that at 90 days, pfa may be used in cast-in-situ concrete piles more effectively. The economic benefit is that, for a typical pile of 5m in length and 0.3–0.4 m in diameter, the materials cost would be reduced by about £1, or 5 to 7%,. For a medium size piling company with an annual pile production of 300,000 linear meters, this would be translated into a saving of £60,000 per year, not a small sum for such little effort.

Durability

Many structures have failed prematurely, not because of inadequate strength, but because of inadequate durability. The cost of the consequent repair work takes up a significant part of the construction expenditure and this is particularly so in developed countries. For instance, it is estimated that in the UK alone, expenditure on repair/maintenance-related problems was in 1993 in the region of £20 billion, or 42% of the total construction output[7]. So, what shall we do?

Understanding of potential deterioration

Many concrete durability problems are due to inadequate consideration at the design stage. Lack of good investigation during the design phase is frequent. Such an investigation should relate to the environmental and exposure conditions and the required material properties. It is important that a designer is made aware of concrete durability problems right at the beginning of the design stage and seeks specialist input where appropriate. The potential deleterious mechanisms have to be identified and that should take into account the details of construction, exposure conditions, the concrete materials themselves and the protection offered to the steel reinforcement. Based on such detailed analysis, appropriate counter measures can then be formulated to delay or to eliminate the process of deterioration.

A structure should be made easy to construct and care should be taken in reinforcement detailing to avoid complexity in construction, as this often leads to problems. For example, over-congested reinforcement will lead to difficulty in vibrating the concrete and hence produce a less dense, less durable mix.

Reinforcement

Steel reinforcement is often taken for granted in our structural design. However, do we really need steel in every situation? The fact that some Roman concrete structures have lasted for thousands of years suggests that concrete structures would be better without reinforcement as far as durability goes. The single most important cause of deterioration of concrete structures is due to the corrosion of steel reinforcement[8]. In some parts of the structures, e.g. mass concrete foundations, the use of steel reinforcement can be completely avoided. Despite much successful experience, many contractors, clients and statutory authorities feel uneasy about this option and further education may be required to convince people that, where only compressive loads are applied, reinforcement is not needed.

In most instances, the use of reinforcement is probably justified. However, if the environment is aggressive, we can consider increasing the level of protection. For a start, the depth of concrete cover to the steel can be increased. There are special types of reinforcement which may be used, such as epoxy-coated rebars, non-ferrous fibres or reinforcement. If the environment is really aggressive, consideration can even be given to the installation on the structure of a cathodic prevention or cathodic protection system as a further preventive measure.

Characteristic cover depth

In designing concrete structures for durability, the depth of reinforcement cover is absolutely crucial. With a chosen concrete mix and given exposure conditions, it is the thickness of this cover that determines the time to initiation of corrosion of steel rebar. However, very seldom is the specified cover depth satisfactorily achieved in practice. A survey(9) carried out in the UK on 200 concrete bridges showed inadequate cover is commonplace. For bridges built from 1960 onwards, the minimum requirement of BS 5400 for C40 concrete had not been met in 46% of the bridge supports and 27% of deck soffits. Another study(10) in the Middle East on 42 reinforced concrete framed structures, of an age 15 to 20 years, showed that in most cases corrosion and consequent spalling could be related to inadequate cover. In these surveys, cover was reported to be less than 12.7mm in 68 per cent of the observed spalls.

A similar study carried out by the authors on a number of major structures in the Far East arrived at a similar conclusion. Figure 2 shows the result of a typical survey, comprising some 1400 measurements. The specified nominal cover depth was 60mm with tight than usual permitted deviation, but some 40 per cent of the measurements were found to be below the minimum acceptable cover depth.

In view of what may be reasonably expected in practice and the importance for provision of sufficient cover, measures should be taken to ensure that minimum cover is actually achieved. One way of doing this is to increase site supervision but, even so, problems are still likely to occur from time to time. The other method, perhaps a more realistic one, is to add a margin to the specified minimum cover depth. With analogy to the design for strength, one can adopt the concept of characteristic cover depth and target cover depth.

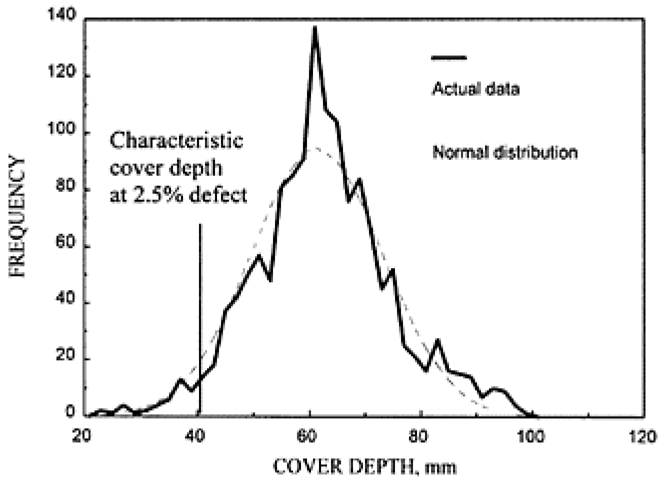

Figure 2 Cover depth distribution

As a first approximation, it may be assumed that the concrete cover depth follows a Normal distribution, as indicated in Figure 2 (For lower nominal cover the distribution curve may become positively skewed). So there is a probability, however remote, that a result will be obtained which is less than the specified value. It is therefore sensible not to specify a minimum cover depth but a "characteristic cover depth", below which a specified proportion of the test results, or the "defect", may be expected to fall. The characteristic cover depth may be defined to have any proportion of defects. BS 8110 adopts 5% defective level for strength. However, it provides a further safeguard against failure by providing a relatively high partial material factor of 1.5. Given the consequences of low cover and lack of other protective measures, a defect level of 2.5% is considered as appropriate.

As a result of the variability of site practice it is necessary to design concrete elements to have a mean cover depth greater than the specified characteristic cover depth by an amount, i.e. the margin. Thus:

$$d_m=d_c+k\times s$$

where d_m=the target mean cover depth

d_c=the specified characteristic cover depth

s=the standard deviation

k=a constant

The constant k is derived from the mathematics of the Normal distribution and increases as the proportion of defects is decreased. For percentage of defects of 10, 5, 2.5 and 1% respectively, its value is 1.28, 1.64, 1.96 and 2.33.

The value of s depends on many factors and investigations carried out by the authors showed that it broadly related to the mean cover depth. The results are shown in Figure 3. For mean cover depth of 25mm or less, s may take the value of 5mm. For a mean cover depth greater than 25mm, s is approximately 20% of the mean cover depth.

Hence, for a specified characteristic cover depth of 30mm and 97.5% probability of exceeding this value, the target mean cover depth can be worked out as below:

d_m=30+1.96 (0.2×30)=42 mm

Clearly this concept needs further investigation, particularly with respect to the standard deviation of cover. Consideration also needs to be given to the type and nature of construction, the level of supervision and applied quality control.

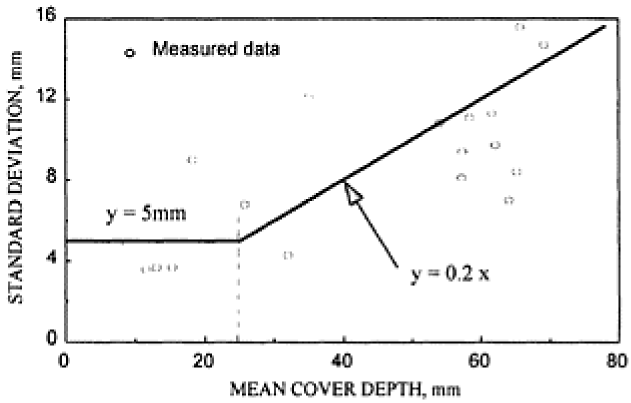

Figure 3 Cover depths vs. std deviation

Surface protection

Appropriateness in concrete design means the recognition in a positive manner of the merits and limitations of the materials. Protection to concrete structures under some circumstances is not only desirable but essential. The recently completed Shantou Harbour Bridge in southern China is perhaps the longest suspension bridge using a concrete deck, with a centre span of 450 metres. Figure 4 shows the bridge during construction. For such a long bridge deck, self weight was a major problem and in order to minimise it, a combination of high performance concrete and a small cross section had to be employed. With the small cross section, the effective cover depth at the soffit of the deck was only 18mm and, since the bridge is exposed to a semimarine environment, its durability was a serious concern. To compensate for the low cover depth and to help achieve the design life of 120 years, it was decided by the engineers involved in the design and construction of the bridge to provide a surface coating to the structure. In this particular case, a high performance, composite epoxy coating system was selected, which provides excellent resistance to ingress of both carbon dioxide and chloride ions. It has a design life in excess of 10 years, after which recoating may become necessary.

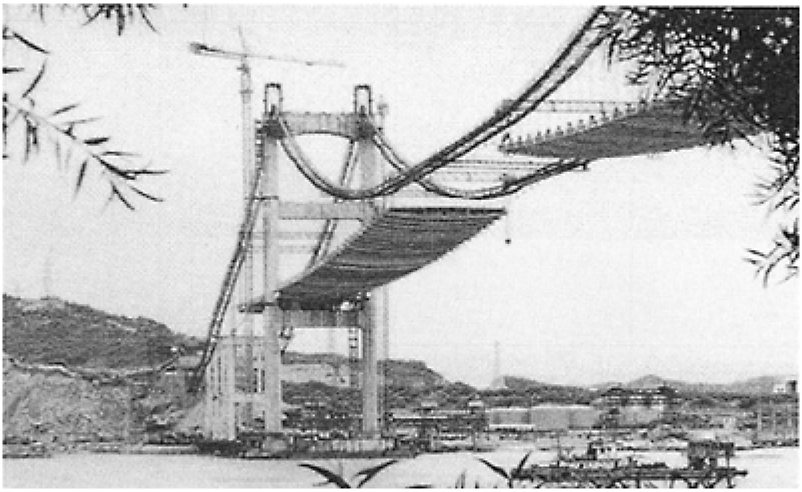

Figure 4. Shantou harbour bridge under construction

Practicality

Experienced engineers will know that it is no better to have a theoretically perfect, but, impracticable, design than to have a bad one. It is therefore important that, in carrying out the design, the practicality is carefully considered. This includes the availability of materials, their quality and consistency, the availability of a trained workforce, quality control and the construction conditions.

At present the use of high strength high performance concrete (HPC) is gaining popularity in Hong Kong. Grade 100 concrete mix was used, for the first time, on a commercial project last year[(11)]. Along with its use, there were many problems which had to be resolved. For instance, while most concrete constituents were obtained locally, silica fume, which was required for the mix, had to be imported. This meant that, before the use of HPC, appropriate and sufficient storage of silica fume at the site of the ready mix supplier had to be resolved.

Quality procedures have to be tightened to cope with the smaller tolerance of the concrete mix. The accuracy of batching and weighing machines has to be frequently checked and maintained, as small changes in water or cement measurement will result in a relatively large variation in water-cement ratio and, hence, the strength. For instance, within the normal accuracy of a balance, a variation in weighing cement or water of ± 2kg may result in a change in strength of 3–6N/mm^2 for a grade 100 mix compared with only 1–3 N/mm for a grade 40 mix. As the total water content of the mix is less, any loss in evaporation will have a detrimental effect on the properties of the concrete, especially that near the surface. This requires that special care is taken in the curing of the concrete and water loss from concrete is minimised.

With the use of HPC, the concrete mix tends to have a higher cementitious content. The amount of heat generated during cement hydration and, hence, the temperature rise of the concrete can be very significant. For a typical Grade 100 mix and a concrete section size of 2 meters, if no special measures are taken, concrete temperature can rise by as much as 70°C during the first 24 hours after casting. With such a steep rise in temperature, thermal cracks can develop and the high temperature could also lead to the development of a deleterious mechanism termed delayed ettringite formation, which in the long term leads to cracking of the structure(12). Both these factors could have a detrimental effect on the durability of the concrete structure and, therefore, require careful consideration at an early stage for their avoidance.

Economy

In the search for a technical solution, an engineer has a duty to ensure that his design meets the requirement of the client and is economic. He will have to consider the cost of the materials, plant, labour and also the quality of the end product. One thing worth bearing in mind is that an economic solution does not necessarily imply a cheaper initial installation. The whole life costing of the structure also has to be considered. This enables the construction and operation costs over the life of a structure to be expressed in present values and different strategies to be compared on a common basis. Substantial savings may be achieved by ensuring durability at the design and construction stages.

It has been suggested that there is a “law of five”(13) which states that “one dollar spent in Phase A(see Figure 5) equals five dollars spent in Phase B equals twenty-five dollars spent in Phase C and equals a hundred and twenty-five dollars spent in Phase D”. What this means is that a little extra attention to durability during design and construction (say one dollar) can lead to substantial gain with respect to renovation cost (of say 125 dollars).

Again taking the Shantou Harbour Bridge as an example. The decision to protect the bridge deck with a high performance epoxy coating system itself is based on the consideration of minimising the whole life costing of the bridge. Having selected the main component of the coating as being epoxy, a further question is the selection of the top coat. There were a number of choices, including chlorinated rubber, acrylic and polyurethane. From weathering tests conducted on these specific products, it was found that chlorinated rubber offered the least resistance to weathering and the polyurethane top coat was some 50% more durable than the acrylic one. As the bridge is to be operated as a toll bridge, it was obviously in the project’s interest to maximise the maintenance period, which could further reduce the associated costs in materials, labour, access provision and, more importantly, lane possessions of the motorway. Therefore, although the cost of polyurethane top coat was almost 100% more expensive than its acrylic counterpart, it became the most favoured candidate for application on the bridge as the top coat.

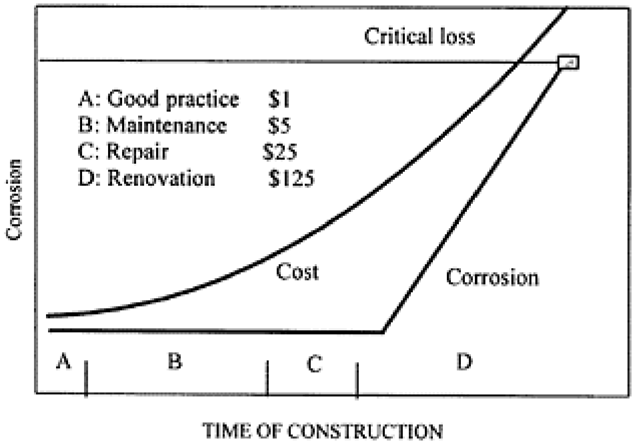

Figure 5 The "Law of Fives" (Ref: 13)

Other Considerations

Engineers have a responsibility to the society in which they live. In this respect, they have to consider other aspects of the concrete technology, e.g. the law and regulations, health and safety in the use of the materials, environmental protection etc. Of particular relevance may be the protection of the environment. When using concrete technology, engineers should consider how the use of waste materials from other industries will benefit not only concrete technology but also the environment. The use of pfa and ground granulated blastfurnace slag will reduce the need for costly disposal of these materials and will also reduce the resources and energy needed to produce cementitious material. These materials, when used judicially, have the added benefit of improving concrete durability.

In order to maximise the benefits of employing appropriate concrete technology, efforts should be made to reduce regulation and over-specification of concrete to permit innovation.

CRITERIA FOR APPROPRIATENESS

Based on the above discussions, the authors would like to propose the following four main criteria for assessing the appropriateness in concrete design.

First and foremost, a concrete design should be assessed on the basis of fitness for purpose. This includes the engineering properties as well as the durability aspects of the materials, taking into consideration what is desirable and what is actually achievable in practice.

Secondly the design should be practicable, in terms of material availability, labour skills, level of quality control, requirements for plant and equipment. Over complexity should be avoided as it often leads to construction problems.

Thirdly, the design should be an economic one. This should consider not only the cost of initial installation, but also future requirements for maintenance and repairs, which in many cases could considerably outweigh the initial cost.

Finally, the design should be socially and environmentally sound, taking into account the requirements from the law and regulations, health and safety and impact on the environment.

ACKNOWLEDGEMENT

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ASSURING PERFORMANCE OF CONCRETE STRUCTURES THROUGH A DURABILITY AUDIT

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ABSTRACT. This paper describes a Durability Audit under development by the Building Research Establishment in collaboration with UK industry. The aim of this audit is to ensure that concrete structures are durable for their required service lives in the environment to which they are exposed.

The Audit will require demonstration that durability has been taken fully into account at each appropriate stage, from specification of the required service life by the client, through characterisation of the environment, design for durability, planning of maintenance and specification of materials, to quality of execution. It will not, in itself, dictate the required level of durability or how it is to be accomplished. It will thus not affect the flexibility or freedom of the client in specifying his needs or of the designer in finding the best solution for each situation. The Durability Audit has the potential to play a major role in assuring the durable performance of future concrete construction and so reduce the need for concrete repairs.

Keywords: Audit, Brief, Construction Process, Design, Design Life, Durability, Execution, Service Life.

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INTRODUCTION

A strategy, entitled 'Durability by Intent', has been developed by the UK Department of the Environment, in conjunction with the Building Research Establishment (BRE) and the UK construction industry, for its programme on concrete and reinforced concrete. The aim of this strategy is basic:

> 'To ensure that concrete is durable for its planned life'

This paper describes the concepts of a Durability Audit currently under development at BRE in order to enable the construction industry to achieve this aim. The paper identifies the different stages of the construction process and how they affect durability, previews how the audit might operate and predicts its likely impact on the durability of future concrete construction.

Premature deterioration of concrete and reinforced concrete in structures has been a matter of serious concern for several decades and remains so at present. Although some of the recent changes in the use of materials may have a limited impact on the general level of durability, little of fundamental significance would seem to have been applied across the construction industry that is likely to cause a radical improvement in durability. For example, a recent as yet unpublished study of the achievement of cover to reinforcement in 25 structures, under construction in the UK during 1993/94, showed that failure to achieve specified cover is still commonplace. Inadequate cover is, of course, a major contributory factor in the deterioration of concrete through reinforcement corrosion. It is thus likely that a significant proportion of concrete structures currently being built will lack durability in the future, thus justifying the introduction of radical measures aimed at tackling the whole problem.

THE CONCEPT OF A DURABILITY AUDIT

The Durability Audit is defined as:

> A system that identifies responsibilities for durability at all stages of the construction process, and requires demonstration that durability has been fully taken into account at each stage.

It will be a formal system in which the decisions and actions affecting durability will be examined at all stages of the construction process. The principle stages of the construction process and how they affect durability are considered in the next section. If adopted, the audit would most likely be required by the client for the structure; it would comprise a scrutiny of appropriate evidence of how durability has been taken into account. This evidence would be provided by the party responsible for the stage of construction under consideration.

It is a matter for consideration whether the system would be self-certifying or third party-certified. Whichever route is taken there would need to be consensus on how to define the need for durability and how to demonstrate the need was satisfied.

Depending on the perceived risk, third party assessment could be needed at each stage of the construction process and before the next stage was allowed to start. For example, it may be considered essential to complete the audit of the client's brief before detailed durability design can take place. Likewise, the detailed durability design may need to be audited before construction of that part of the structure can commence.

The audit is intended to ensure that any potential problems with respect to durability have been identified and resolved before that stage of the construction process is commenced. It is not, however, the function of the audit itself to identify errors or problems; it should be a confirmation that durability has received the required explicit consideration. Identification by the audit of residual errors or problems will result in failure of the audit or qualification of the audit report, either of which would be a matter of serious concern.

The audit will not, in itself, restrict the freedom or flexibility of the designer's approach to achievement of durability in the structure. The audit will also not dictate the level of durability that is required; that would be a feature of the client's brief. The required level of durability, however, will be the 'standard' against which the design, specification and execution will be audited.

Although initially it will be restricted to concrete and reinforced concrete, it is intended to produce an audit system which will cover the durability of all materials in all types of structures. Restriction to concrete at this stage will, however, ensure that rapid progress is made in development of the system. The broad principles developed for concrete structures will then be applied to other materials and other types of structure.

The authors believe that the design for durability of concrete and reinforced concrete is a sufficiently complex process to justify the initial restriction of the audit to these materials. Much of the concrete used in structures is specified individually for particular projects to deal with widely varying exposure conditions. For many of those conditions, particularly exposure to chlorides, there is no sufficiently comprehensive database of known performance to identify 'standard service lives', whereby elements used in certain generalized exposure environments may be expected to last for a given period with a reasonable degree of certainty.

The concept of a durability audit is neither new nor unique to BRE. There is already in operation a highly succesful technical audit system, which includes consideration of durability, run by Housing Association Property Mutual Ltd (HAPM) in the UK for the insurance of Housing Association properties. Through this system HAPM audits the design, drawings and specifications of all property to be insured in order to identify departure from good practice. Components of the buildings are assessed against a large database of known performance [1] to assign 'insured lives' of up to 35 years. This database includes several concrete components such as pre-cast floor beams, in-situ lintels, drain pipes and paving. The scheme under development at BRE is not intended to be linked directly to insurance cover and thus will be based on 'required service life' rather than 'insured life'. Nevertheless, assessment of the risk of premature failure will play an important part.

DURABILITY CONSIDERATIONS AT THE DIFFERENT STAGES OF THE CONSTRUCTION PROCESS

To ensure the effectiveness of the Durability Audit, it will cover all the decisions and activities that have an influence on the durability performance of the structure, at all stages of the construction process. The main stages are shown in figure 1.

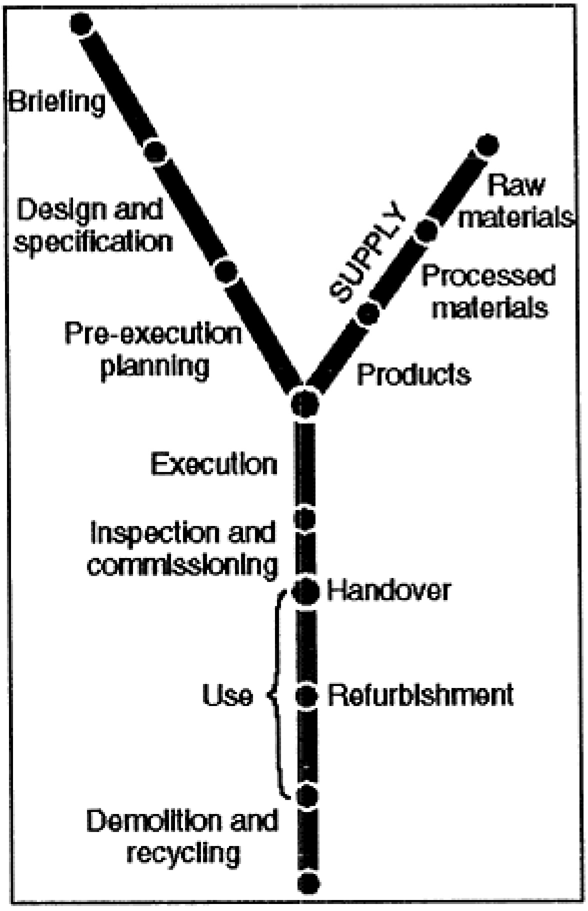

Figure 1 Simplified model of the construction process

Some of the principle questions that will be addressed by the audit at each stage, are given below:

Briefing

Have the client's requirements for durability been established and clearly stated in terms of required service life and acceptable maintenance levels? Has an adequate construction period been allowed with regard both to duration and to likely weather conditions?

Design and Specification

Does the design concept introduce unusually severe durability conditions? Have the ground and climate-related exposure conditions been accurately assessed? Has an appropriate durability strategy for resistance of these conditions been selected? Has the structure been designed for a life at least as great as the required service life? Are components where the design life is less than the required service life easily replaceable? Is the structure buildable as designed and is the design free of detailing errors? Have essential maintenance requirements been identified and a maintenance plan produced?

Pre-Execution Planning

Has a sufficient time been allowed to ensure all operations can be performed to the standard required, thus ensuring the structure can be built as designed? Has consideration been given to ensuring that forseeable weather conditions do not reduce the quality of the construction?

Supply of Materials and Products

Are all materials and products in accordance with the specification and of the required quality to ensure the design life can be achieved?

Execution

Has the structure been built as it was designed, with a sufficient level of quality to ensure it will be able to last for its design life? In particular, has the concrete been adequately compacted, finished and cured and is the cover to reinforcement within specification?

Commissioning and Inspection

Has the predicted service life been determined from the quality of the completed structure and compared with the design life and the required service life? If the predicted service life is less than the design life, has appropriate remedial action been taken? Predicted service life could be re-examined after a 'durability reference period' of, say, ten years [2–4]. For critical components, can the concrete quality be measured before the next stage of construction commences?

Use

Are the planned inspections and maintenance being carried out at the required intervals and to the required standard? Is the structure performing as designed?

Modernisation

Has the effect on the durability performance of the structure of any change of use been assessed in relation to the required service life?

Demolition and Recycling

Although not strictly part of the construction phase affecting durability, has consideration been given to the ease of demolition, and to the re-use of materials to as great an extent as is practical and economic?

AUDITABLE ITEMS

The contents of the client's brief can be assessed against a list of essential requirements based on BS7543 [5] and/or the draft checklist produced by the UK Construction Industry Board Working Group 1 in conjunction with Reading University. The brief, particularly the statement of required service life, will then form a principal standard against which subsequent decisions and actions are audited.

The design can be audited against the client's brief and the principles of the draft European Standard for the basis for design [6]. Auditing of the detailed durability design will depend on the basis upon which the design was produced. The flexibility of the audit system will ensure that no constraint is placed upon the designer to choose a particular durability strategy or durability design method. For example, design could be based on a particular deterioration model provided it can be shown that the particular model is appropriate for the exposure conditions to be resisted and that the validity of the model can be demonstrated. This, of course, may cause some problems in the early days of operation of the system but, hopefully, no designer would choose to use a particular model unless its validity can be readily demonstrated.

Delivery tickets for materials and products can be audited against design specifications.

Execution and execution records (inspection records, delivery tickets for materials, etc) can be audited against the designer's drawings and specifications.

Records of maintenance can be audited against the designer's maintenance plan.

DEMONSTRATION OF COMPLIANCE

The audit will require demonstration of compliance of decisions and actions with appropriate and acceptable standards, some of which have been suggested in the preceding section. The acceptability of the standard with which compliance is demonstrated will be assessed by the person or body responsible for the audit. Acceptability of an ’audit standard’ can be by general consensus (e.g. BRE Digest, Concrete Society Technical Report) or it may have nationally accepted status as a National or European Standard or Code of Practice. Some audit standards, such as innovative design models, may need to be assessed on an individual basis.

A not insignificant problem arises through the general absence from National Standards and Codes of Practice of any specific indication of expected service life. The British Standard for the structural use of concrete, BS8110 [7], for example makes no mention of service life or design life. The handbook to the standard [8], published in 1987, makes the following statement:

> ’As with other structural materials, knowledge is not yet adequate to allow concrete structures to be designed for a specific durability and life. Structures designed and built according to the recommendations in the Code may normally be expected to be sufficiently resistant to the aggressive effects of the environment that maintenance and repair of the concrete will not be required for several decades, i.e. a life before significant maintenance generally in the region of say 50–100 years.’

British Standard BS5502 : Part 22:1993 [9] for the design of buildings and structures for agriculture is a notable exception in that it gives values of concrete quality (minimum cement content and maximum water/cement ratio) and cover to reinforcement for stated durability lives of 10 and 50 years for concrete containing reinforcement, and 20 and 50 years for unreinforced concrete.

It is thus clear that improved guidance is required from National Standards and Codes of Practice on expected service lives from different concretes in different environments. Such change is, however, likely to be slow to develop and may require extensive review of existing information on performance in service. In some cases it may be necessary to commission new research.

Successful application of the Durability Audit will depend, for some parts of the construction process, on scrutiny of Quality Assurance records so that the auditor does not have to, say, perform a covermeter survey of the structure to check compliance with the specification.

Successful demonstration of compliance in the audit will result in the issue of an unqualified audit report. This report will be a statement that, in the opinion of the auditors, the records of decisions and actions related to durability demonstrate an acceptable probability of achievement of the (client’s) requirements for the structure.

A qualified audit report would be issued where full compliance cannot be demonstrated. A qualified report must command the status of serious concern to the client for the audit. Nevertheless, only something important will cause auditors to quality their report. The qualification might be due, for example, to a lack of proper records or disagreement on acceptability of any ’standard’ against which a durability related decision or action has been made.

A TRIAL RUN WITHIN BRE

Further development of the system is being enabled by application of some of its main principles to the construction of a new office and lecture theatre complex on BRE's Garston site. This building comprises a reinforced concrete frame and precast vaulted concrete floors with an in-situ concrete topping. It also features an in-situ corridor slab of 100mm thickness and a 250mm thickness in-situ slab as part of the floors in seminar areas. Foundations are a ground supported pad in class 2 sulfate soil. Much of the in-situ concrete will contain recycled aggregates.

Much of this building is being constructed from materials other than concrete, such as the stainless steel roof, the brick façade, timber window frames and some timber wall-cladding, but the audit trial will initially be limited to the concrete components.

OTHER CONSIDERATIONS

It is important to minimise the extra workload resulting from the audit; although, even in the short term, audits should reduce the overall workload simply by clarifying the requirements at each stage of construction. As far as possible, therefore, it is desirable to integrate the Durability Audit with other initiatives such as energy and environmental assessment, CDM (Construction, Design and Management) regulations for safety, and life-cycle costing to reduce the burden on designers and constructors, etc. For example, there is a considerable amount of overlap between the information required by the Durability Audit and that contained in the H&S file required by the CDM regulations.

Application of the Durability Audit will initially be on a voluntary basis, probably at the request of the client. Nevertheless, for some types of structure difficulty might be experienced in encouraging the use of the system, for example, where the client is a developer whose financial interest will cease once the structure, or full-repairing lease, has been sold. The party who, in that case, will bear the financial liability for inadequate durability will have had no involvement in the briefing, design or execution stages, but would clearly stand to benefit from ownership of a structure where the audit system has been used.

FURTHER DEVELOPMENT OF THE SYSTEM

Development of the system will include the necessary gathering of information on many aspects of the construction process. This will include the procurement of data on the normal variation of properties of concrete in structures. For example, cover to reinforcement for in-situ concrete in the UK is commonly specified, in accordance with BS8110 [7], as a nominal value with a negative tolerance of 5mm to allow for the variability of fixing. Many studies have been made of cover achieved in practice [10] and it has generally been shown that the maximum deviation from the specified nominal cover is commonly significantly greater than the tolerance of 5mm. A durability design that relies on achievement of a minimum cover of the nominal cover less a 5mm tolerance is therefore flawed; this problem will be identified within the Durability Audit. If, however, it can be demonstrated that special measures will ensure that a specified 5mm tolerance will be achieved in practice, the audit will accept this.

There are few, if any, structures where concerns about durability can be limited to a single material such as concrete. It is intended that the Durability Audit will, in due course, be extended to cover all materials and components within a structure.

An audit administration body could be established, the functions of which would include advising clients on the appointment of auditors and deciding on the acceptability of ’audit standards’ such as guidance documents, design models and codes of practice, and the eligibility of evidence such as Quality Assurance records. In many ways the concept of the audit parallels the BREEAM methodology, run by BRE for the environmental assessment of buildings, and BRE could provide the ‘ownership’ function whilst industry undertook the auditing.

BENEFITS LIKELY TO BE OBTAINED

The primary benefits of the Durability Audit will be the reduced incidence of repair, reduced lifetime costs and reduced disruption of use due to unforseen maintenance or repair. A successful outcome, i.e. an unqualified audit report, will provide clients with greater assurance that the structures they have commissioned will perform as required, and without significant unplanned expenditure, during their required service lives.

Acceptance of the audit system could provide a valuable guide for insurers in assessment of their risk. A favourable audit report should allow the insurer to offer significant discounts in reflection of the reduced risk.

The need for definitive guidance on design life and durability performance will provide the incentive for more extensive continuous review of standards and codes of practice than is currently the case.

Development of this Durability Audit will identity areas where essential work is needed to provide guidance on durability design or specification for a given design life, against which an audit can be performed.

CONCLUDING REMARKS

This paper has described a proposed Durability Audit and how it can play a major role in assuring the durability performance of future concrete construction. Indeed, the principles involved extend well beyond just concrete construction and, in due course, the audit system can be extended to include all construction materials and construction types.

Introduction of the Durability Audit will encourage the client, designer and constructor to take a much harder look at building durability into their structure than may commonly be the case at present. It is acknowledged that within many prestige projects, provision of durability is already receiving extensive consideration. This is to be commended but, hopefully, the Durability Audit will encourage such detailed consideration to be extended to ’normal’ structures. BRE is ready to play its part in developing the scheme together with industry.

ACKNOWLEDGEMENTS

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REALISTIC PREDICTION OF TIME-DEPENDENT RESPONSE OF CONCRETE STRUCTURES

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ABSTRACT. Strict demands of investors to shorten the constructional time can often be satisfied only by using special building technologies which enable quicker finishing of construction and certain rationalizations, but also demand special technical treatment. The paper presents a method of general non-linear analysis of time-dependent response of concrete structures. It is limited to reinforced, prestressed or composite plane frames. In addition to the influences of rheology, including nonlinear creep of concrete, the timevariable structural system can also be taken into account. Therefore the presented method of analysis is suitable for predicting the time development of displacements and internal forces, and making an estimation of the safety of structures in any intermediate or final constructing phase. Starting from theoretical foundations appropriate software has been elaborated for numerical analysis of structures. It is, considering the complexity of data preparation, equal to the usual software for nonlinear static analysis of concrete frames by means of the finite element method. The applicability of the prepared software is briefly presented by examples.

Keywords: Aging, Building Technologies, Changeable Structural System, Nonlinear Creep, Nonlinear Analysis, Rheology, Relaxation, Shrinkage, Time-dependent Response

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INTRODUCTION

Increased demands for shortening the constructional time often have adverse effects on the quality of structures. Strict demands of investors can frequently be satisfied only by using special building technologies which enable faster finishing of construction and certain rationalizations, but also demand special technical treatment. With certain technologies or construction sequences (e.g. free cantilever construction of bridges) also the assurance of adequate final geometry of structure can present a special problem. Almost all structures, apart from the simplest ones, are constructed gradually. Besides material and geometrical nonlinearities, usually included in a computational analysis, their gradual construction is another important contribution to nonlinear behaviour of structures. In that case a part of the structure is loaded and deformed even before the completion of the whole structure whereas the structural system can be changed basically. In reinforced or prestressed concrete structures additional handling is necessary due to rheology of material. Owing to the fast progress of works there is often a desire to prestress and de-shutter the structures soon after the concreting. During prestressing the stress level can be higher than later, during the normal use of the structure. Furthermore, the concrete is still young in this period, which increases the initial and time-dependent strains due to rheology. Rheology causes in concrete structures time changes of displacements, whereas in statically indeterminate structures it also causes redistribution of internal forces. Stress-independent rheological phenomena are aging and shrinkage of concrete, stress-dependent ones are creep of concrete and relaxation of steel. Within the range of working stress creep of concrete depends linearly on the stress, whereas at higher stress levels this dependence becomes explicitly non-linear.

There are relatively few theoretical papers which deal with the problem of rheology in conjunction with the variable structural system, and there are even fewer presentations of appropriate experimental researches. The problem is in most cases solved by generalized method of forces where creep of concrete is only considered according to linear theory. Such an approach naturally brings up to the system of Volterra integral equations for unknown forces which can be solved only numerically in general. The complexity of the system of equations depends on the type of structures which can be divided into homogenous and heterogeneous ones [1], with respect to static determination, different ages of individual parts, existence of reinforcement and possible elastic supports. For the realistic prediction of time response each reinforced concrete structure must be treated as a heterogeneous one. In this case, however, the systems of integral equations become so complicated that these methods are actually unsuitable for practical use in civil engineering.

The method of finite elements (FEM) has been applied in the presented method of analysis of time response of concrete structures considering together the rheology, variable structural system, cracks and geometrical non-linearity. For modelling of viscoelastic properties of aging concrete the generalized Kelvin model has been used which can in addition to experimentally acquired functions of concrete creep approximate also creep functions of concrete according to the CEB-FIP 1990 model code [2]. Non-linearity of creep was included by functions which in dependence on the momentary stress level increase the speed of creep [3,4]. For time integration, the method with discretisation of inelastic strain, considering the linear interpolation of stress within the time interval, as presented by Bažant [5], has partly been modified. As a basis for software development the NONFRAN computer program [6] has been used which is intended for general static analysis of planar frames considering material and geometrical non-linearity. The elaborated software allows consideration of gradual changing of structural system through constructing phases. Graduallity of constructing can be simulated on the level of whole elements or on the level of cross-sections consisting of subsections.

MODELLING OF MECHANICAL PROPERTIES OF MATERIAL

Constitutive Law of Concrete

In the analysis of concrete structures by means of the FEM with rheology being considered incremental constitutive laws of concrete are most often used. They can be expressed by Maxwell or Kelvin chains respectively. In the presented method of analysis both kinds of models can be used; later on, only the model of Kelvin chain is presented due to possible direct determination of the model's parameter on the basis of experimental results of creep or creep functions of concrete according to different code-type formulations.

Time domain can be divided into discrete times 0, t_1, t_2,…, t_{r-1},t_r,…, which represent limits of time intervals $\Delta t_r=t_r-t_{r-1}$. For time integration the modified step-by-step method of integration is used with discretisation of inelastic strain by linear interpolation of stress in a time interval: $\sigma(t)=\sigma_{r-1}+ (t-t_r)\cdot\Delta\sigma/\Delta t_r$, which was in its basis presented by Bažant [5]. In an arbitrary time step Δt_r there is a relationship (1) between the increment of strain $\Delta\varepsilon$ and the increment of stress $\Delta\sigma$. The elements in the brackets represent nonlinear initial strain and creep strain due to the stress increment $\Delta\sigma$, the third element represents the influence of loading history.

$$\Delta\varepsilon-\Delta\varepsilon_o=\Delta\sigma\left[\frac{1}{E(t_{r-1/2},\sigma_{r-1},\Delta\sigma)}+f(\sigma_{r-1/2})\sum_{i=1}^{n}\frac{1-\lambda_{i,r}}{\hat{E}_i(t_{r-1/2})}\right]+f(\sigma_{r-1/2})\sum_{i=1}^{n}\dot{\varepsilon}_i(t_{r-1})\Delta t_r\lambda_{i,r}\cdot \qquad (1)$$

$$\dot{\varepsilon}_i(t_r)=\dot{\varepsilon}_i(t_{r-1})+\left[\frac{\Delta\sigma}{\hat{E}_i(t_{r-1/2})}-\dot{\varepsilon}_i(t_{r-1})\Delta t_r\right]\frac{\lambda_{i,r}}{\tau_i} \quad (2); \qquad \lambda_{i,r}=\frac{1-e^{-\Delta t_r/\tau_i}}{\Delta t_r/\tau_i} \qquad (3)$$

Indices r-1/2 describe the values of quantities in characteristic time $t_{r-1/2}$ of interval, $E(t_{r-1/2}, \sigma_{r-1}, \Delta\sigma)$ is stress dependent secant elastic module in the time $t_{r-1/2}$, $\hat{E}_i$ is parameter of i-th Kelvin unit, n is the total number of Kelvin units, $\Delta\varepsilon_o$ is a stress-independent increment of strain due to shrinkage of concrete and temperature influences, $f(\sigma)$ is function which represents the influence of nonlinear creep, $\dot{\varepsilon}_i$ is the hidden variable of i-th Kelvin unit (time derivation of deformation) being for the time t_r defined by the equation (2). τ_i is the retardation time of i-th Kelvin unit.

As the function $f(\sigma)$, representing the influence of nonlinear creep in dependence on stress level $\overline{\sigma} = |\sigma| / f_c(t)$, the formulation (4) from the CEB-FIP MC 90 [2] can be used, being applicable to the stress level $0.4 \leq \overline{\sigma} \leq 0.6$, or the suggestion (5) or (6) by Bažant, Prassanan and Kim [3,4], which can also be considered as a rough estimation in the area of higher stress, $f_c(t)$ is the average compressive strength of concrete in time t.

$$f(\sigma) = \exp[1.5 \cdot (\overline{\sigma} - 0.4)] \quad (4); \qquad f(\sigma) = \frac{1+\overline{\sigma}^2}{1-\overline{\sigma}^{10}} \quad (5); \qquad f(\sigma) = \frac{1+3\cdot\overline{\sigma}^5}{1-\overline{\sigma}^{10}} \quad (6)$$

When concrete is subjected to sustained high compressive stresses the strength decreases with time of the load. In the computational analysis of structures reduced compressive strength of concrete is also considered due to previous sustained loads. Additionally, a special algorithm describes the behaviour of concrete during cyclic loading in time after appearance of cracks, when the concrete is capable to take over only compressive stress.

In the analysis the strain approach is used, where the change of stress $\Delta\sigma$ is determined for defined increment of deformation $\Delta\varepsilon$. Equation 1 is written as a function of the change of stress in the form $g(\Delta\sigma)=0$ and solved iteratively by improved secant method. If the formulation of creep from the CEB-FIP MC 90 [2] is taken as the basis, the parameters of Kelvin units can be defined as follows. The function which describes the development of creep $\beta c(t-t_0, \beta_H)$ (eq. 2.1–70 [2]) is developped into Dirichlet series $\Sigma A_i(\beta_H) \cdot (1 - e^{-t/\tau_i})$. Retardation times τ_i are chosen in advance [3,5], coefficients $A_i(\beta_H)$ are determined according to the least square method. By means of this method polynomes, of relatively low degree, can be prepared in advance which allow to approximate accurately enough the coefficients $A_i(\beta_H)$ in the range of expected values β_H. The next step is to define parameters of Kelvin units according to equation (7). Quantities ϕ_{RH}, $\beta(f_{cm})$ and E_{ci} are determined by equations (2.1–66), (2.1–67) and (2.1–16) in CEB-FIP MC 90 [2].

$$\hat{E}_i(t) = E_{ci} \cdot (0.1 + t^{0.2}) / \left[\phi_{RH} \cdot \beta(f_{cm}) \cdot A_i(\beta_H)\right] \qquad (7)$$

The main advantage of the chosen incremental constitutive law of concrete is that only a stress, a situation indicator (possible tension or compression failure) and hidden variables of Kelvin units in the previous time have to be saved for each integrating point. This enables an analysis of large structures at almost unlimited number of time steps also on a PC.

Consitutive Laws of Steel

According to the suggestion by Goldberg and Richard, prestressing steel has always been taken into account only as non-linear elastic material. The influence of relaxation of material due to high stress level, as suggested by Stüssi, has also been included. For ordinary steel reinforcement the bilinear law with strain hardening is used in case of considering the hyperelastic behaviour of material. At a known limit of elasticity $(f_{sy}, \varepsilon_{sy})$ and degree of hardening $E_s h$, the stress σ can be as the function of strain ε written with only one relationship (8) for the whole definition range ($-\varepsilon_{su} \geq \varepsilon \geq \varepsilon_{su}$). For the simulation of elastoplastic behaviour of the reinforcement steel two relativly simple elasto-plastic models have been used, which include kinematic strain hardening or isotropic strain hardening.

$$\sigma(\varepsilon) = f^{*}_{sy} \cdot \left(\left| \bar{\varepsilon} + 1 \right| - \left| \bar{\varepsilon} - 1 \right| \right) + E_{sh} \cdot \varepsilon; \quad \bar{\varepsilon} = \varepsilon / \varepsilon_{sy}; \quad f^{*}_{sy} = (f_{sy} - E_{sh} \cdot \varepsilon_{sy}) / 2 \tag{8}$$

SOFTWARE FOR ANALYSIS OF STRUCTURES

For numerical analysis of concrete frames, where influences of nonlinear creep, shrinkage, aging of concrete, reduction of concrete strength under sustained loads, relaxation of prestressing steel, cracks and geometrical non-linearity of the structure are considered, the FEM is used. Characteristics of the elaborated software could be described as follows:

• Basic equations of the beam element are deduced by means of mixed potential energy functional so that the independent variable is also the axial force of the element [6]. Highly efficient finite element, labelled P_4, with additional degrees of freedom is used, where the curvature of the reference axis is approximated with polynomes of the 4-th degree.

• All required data are simultaneously given in advance for the whole structure and for all analyzed phases. These data are: geometrical description, material properties, time sequence of the construction, environmental conditions and loading history.

• Structures can be analyzed for different types of concentrated and distributed loads and imposed displacements of supports. The calculation also implicitly considers indirect “load” due to temperature changes, rheology and prestressing.

• Cross-sections of the elements are divided into subsections whose width can be linearly changed. Subsections are divided into layers. Reinforcement and prestressing steel are also discretely distributed. Prestressing steel can also be an autonomous part of the finite element.

• Time changing of structural system can be simulated on the level of elements as well as on the level of cross-sections [7], with the possibility of adding or taking away whole elements or individual subsections of cross-sections.

COMPUTATIONAL EXAMPLES

As computational examples two benchmark tests for evaluation of creep and shrinkage analysis computer programs, proposed By Subcommittee 3 of RILEM TC 114 [8], are analyzed. They concern relatively simple structural problems for which usual engineering assumptions may be done. More comprehensive data relating to the analyzed experimental tests can be found in [8]. The tensile strength, shrinkage and creep functions of concrete, used in the analysis, are derived from its compressive strength according to CEB-FIP MC 90 recommendations, without fitting in any way of the model data for the optimization of calculated results.

Benchmark no. 1 consists of seven tests of simply supported reinforced concrete slabs with a span of 310cm. The geometrical description, element model, cross-section and lay-out of reinforcement in the typical cross-section are shown in Figure 1. At the age of t'=28 days the slabs were loaded by up to five different load levels between P=2.89kN and P=15.73kN. Thereafter the deflection was measured for a time span of up to 510 days. During this time the relative humidity was 60% and the temperature 20°C. Mean compressive strength at 28 days was 30.6 MPa. In the analysis the compressive strengths of individual samples were used. Figure 2 presents a comparison of measured and calculated deflections of slabs.

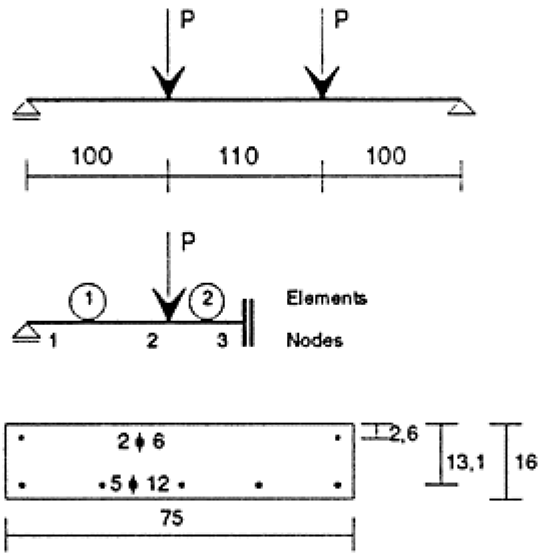

Figure 1: Slabs of benchmark no. 1

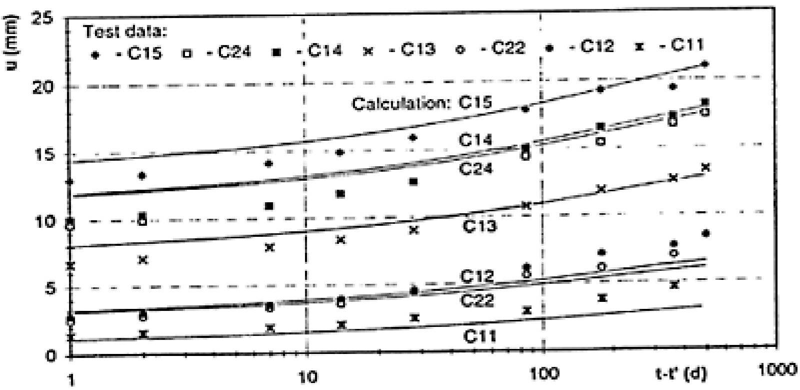

Figure 2: Measured and calculated deflections of slabs

Benchmark no. 2 comprises short- and long-time tests of three slender reinforced concrete columns with rectangular cross-section, built in at one end and free at the other with L=225cm. These tests provide an illustration of the creep buckling phenomenon. The geometrical description, element model, cross-section and lay-out of reinforcement in the typical cross-section are shown in Figure 3. An axial load P is applied at the free end of the columns with an eccentricity e=1,5cm. A load P=280kN was applied to the column II-2 at t'=28 days and sustained for 197 days when failure occurred. Calculations are made for different functions f(σ) (Equations 4, 5 and 6), which represent the influence of nonlinear creep in dependence on stress level. Figure 4 presents a comparison of measured and calculated deflection values of column I-2.

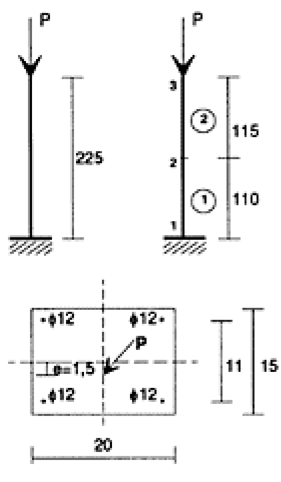

Figure 3 Columns of benchmark no. 2

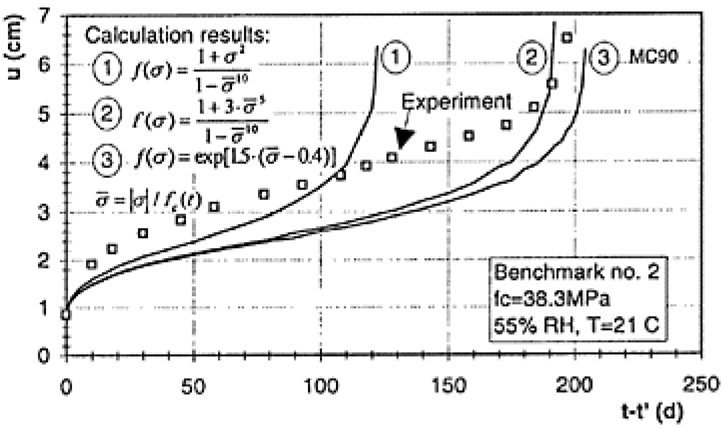

Figure 4 Measured and calculated time dependent 2 deflection values in node 3 of column II-2

CONCLUSION

The presented computational tools and computer software enable an estimation of adequacy of the chosen structural solutions and technology of the construction, as well as the estimation of adequacy of special measures taken during the time of the construction which should provide the anticipated geometry of the finished structure.

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Theme 2
IMPLICATIONS OF HARMONISATION

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Leader Paper

Implications of Harmonisation

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IMPLICATION OF HARMONISATION

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Appropriate Concrete Technology. Edited by R K Dhir and M J McCarthy. Published in 1996 by E & FN Spon, 2–6 Boundary Row, London SE1 8HN, UK. ISBN 0 419 21470 4.

ABSTRACT. Harmonisation envisaged in the Single European Act is taking place. Large sections of the industry in the UK still give the impression of indifference to the impending changes. The paper, which is from a UK perspective, sets out the background and details of the various elements of harmonisation. It concludes by discussing some of the consequences.

Keywords: European Harmonisation, Single Market, Construction Products Directive, Essential Requirements, CE Marking, Harmonised European Standards.

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INTRODUCTION

Mention the word "harmonisation". The reaction of many in the construction industry in the UK will be anything but harmonious. It generally brings out the euro sceptics in us. The common perception is that Brussels is imposing new methods of working, obliging us all to use new codes and standards, new methods of production design and testing. There is a natural reluctance to change but it is generally accepted that an expensive changeover is upon us.

Therefore, for the purposes of this paper, I take the title to mean European Harmonisation, which is slowly but surely creeping on us. It will affect our lives in many spheres including the field of construction. In the area of Building and Civil Engineering alone, there now exist 92 dual Euro/British Standards. The number of British Standards which had been withdrawn as at summer 1995 stands at 24. According to BSI, 1996 is likely to see a major acceleration in the birth of new Euro Standards.

In this sceptical climate, it well behoves us to examine some of the details of what is being proposed and the positive aspects of the impending change.

SINGLE MARKET

The Single European Act came into force in 1987. Under the provisions of the Act, a huge market of some 370 million people was created on 1st January 1993 across Europe. It has no internal borders. Goods, services, people and capital should move freely throughout this internal market.

Some 300 individual measures had to be adopted by the Commission to facilitate this arrangement. They are aimed at:

a. The removal of physical barriers to trade;
b. The removal of technical barriers to trade through the Community's new approach to technical harmonisation and standards policy and the mutual recognition of testing and certification arrangements;
c. Liberalising public procurement;
d. Free movement of labour and professions;
e. Liberalising financial services;
f. Liberalising transport services;
g. Freedom of capital movement;
h. Company Law
i. Intellectual and industrial property; and
k. Removal of fiscal barriers.

Not all the measures proposed are of equal significance for the construction industry. Those which most directly affect the industry include the Construction Products Directive to remove barriers to trade in construction products.

CONSTRUCTION PRODUCTS DIRECTIVE (CPD)

The primary purpose of the Directive is to:

a. establish mechanisms for free trade in products with a view to promoting competition and increasing choice in the market place; and
b. safeguard essential health and safety requirements.

The Directive lists a number of Essential Requirements, which must, subject to normal maintenance, be satisfied by the construction for an economically reasonable working life. These functional requirements concern:

* Mechanical resistance and stability;
* Safety in case of fire;
* Hygiene, health and environment;
* Safety in use;
* Protection against noise;
* Energy economy and heat retention.

Whereas the essential requirements of the Directive are about whole constructions, such as buildings, the purpose of the Directive is to remove barriers to trade in products. Thus there is only an indirect relationship between the essential requirements and the characteristics of products. The Directive provides for the preparation of Interpretative Documents, which would amplify the essential requirements and identify characteristics of products and in some cases determine the performance levels so as to enable (a) mandates for harmonised standards to be issued by the Commission to CEN (European Standards Organisation) and (b) guidelines for European Technical Approval to be given to the European organisation for Testing and Certification. Suppliers of products will, therefore, generally rely on the harmonised standards and European Technical Approvals without having to concern themselves directly with the essential requirements.

C E MARKING

Products that conform to community legislation earn a right to carry CE markings. Community legislation is limited to compliance with essential requirements. The CE Marking thus only demonstrates that the product meets the minimum requirements necessary to be placed on the market.

the same scope is produced. There is likely to be an overlap of 6 months. This is the result of BSI's agreement when it joined CEN and not a requirement of the Commission.

An European Technical Approval is a favourable assessment of the fitness of a product for an intended use, based on fulfilment of the essential requirement. These approvals may be granted on the basis of European Technical Guidelines, which, as with the Standards, will be prepared on the basis of a mandate from the Commission. All member states have designated at least one body

for the purpose of issuing European Technical Approvals. Currently, the only designated body in the UK is the British Board of Agrément.

ATTESTATION OF CONFORMITY

Attestation of conformity is a legal statement that the product conforms to the relevant harmonised specification, coupled with a declaration by the manufacturer that he maintains an appropriate factory production control. The procedures, which are defined in the Directive, range from a manufacturer's declaration to third party testing and/or certification. The Certificate of Conformity will be issued by a designated body in each member state. Notification of a testing or certification body in one state creates defacto recognition by all other member states, thus avoiding duplication and eliminating any need for mutual recognition.

PUBLIC PROCUREMENT DIRECTIVE

Public procurement is a key element of the Single Market, both on account of the scale of expenditure and because of the perceived susceptibility to political or other pressures to favour a domestic supplier or contractor. Also the contract letting process has been made more visible. Public sector specifiers are required to refer to European Standards or European Technical Approvals where appropriate. Thus the product specification developed to support the Construction Products Directive will effectively be mandatory under the Public Procurement Procedures.

WHERE DO WE GO FROM HERE?

Unless there is a significant "U" turn in the UK or EC, harmonisation envisaged in the Single European Act will be realised. The pace of progress is not predictable. Initially it will not be perfect and it will evolve iteratively and refinements will be introduced on the basis experience.

Should the industry commit itself whole-heatedly and influence the process or be indifferent to the substantial changes which are in the offing? Human nature is reluctant to accept change, especially when it involves expense, re-training and giving up some long cherished practices. Surely it makes commercial sense to keep the pain of transition to a minimum and focus on the new requirements, refine them and adapt. It is, however, unfortunate that harmonisation has coincided with the deep recession in the industry thus reducing the necessary resources.

Single Market offers new opportunities and also poses challenges. Just as suppliers in one state look outwards to cash in on this huge market, their counterparts in other member states, could also be targeting their old safe "domestic market". This competition and mixing of technical cultures, will challenge old assumptions and in turn will create potential for innovation.

The advent of harmonised product standards and approval systems must surely make sense to any supplier who trades beyond the confines of his old domestic scene. While the need for homework regarding the new market will still remain, the commonalities of standards will surely reduce some of the burden.

If the product standards assist suppliers, the harmonised Eurocodes will be of immense benefit to designers. They can design projects anywhere in Europe without being handicapped by the lack of knowledge of particular national standards. Clearly they will still be obliged to research the local area. For example, National Building Regulations are not being harmonised and therefore there may be particular requirements to be allowed for in design. Nevertheless, life is bound to be easier.

It has been argued that (a) CPD is only concerned with products (b) therefore only product standards need harmonising and (c) these standards should rely on national codes rather than Eurocodes, for design matters. This approach overlooks some realities (viz):

* National codes across Europe differ considerably in detail. A manufacturer wishing to supply across Europe may then be obliged to produce different products to comply with the different national codes. This cannot be cost effective.

* Products are incorporated into structures. If products are designed to national standards, then the whole structure needs to be designed to the same standards. This is not harmonisation but preserving the status quo and entirely against the spirit of the single market.
* In the UK, BSI and the industry, are unlikely to have sufficient resources to maintain Eurocodes and British Codes. Such limited resources as are available, will be better channelled into achieving satisfactory Eurocodes, which are made sufficiently flexible that they are able to accommodate well established methods.

Another sector which will benefit from the harmonised Eurocodes, is the software and publishing industries. This arises from the size of the potential market for their products (eg) design aids. This will only be possible if all States work on the same design code.

Regulatory authorities could be confronted in the future, with unfamiliar or an unusual designs and details, on which they will need to pronounce. This is because engineers and manufacturers will come up with different solutions even when working to the same Code. The solutions will be largely based on the technical culture in which they are rooted.

Summarising, the question is no longer whether there should be harmonisation. Politicians have decided that it should take place. The current debate is all about details. It therefore, seems sensible to play an active part and influence the debate. The evolution is unlikely to be smooth or cheap; but it seems inevitable.

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APPLICATION OF EUROPEAN CONCRETE STANDARDS

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ABSTRACT Due to the very specific design-practices which are related to the cultural, economic and educational background of European countries, developments in the field of concrete technology have taken place mainly on a national scale. With the introduction of the Eurocodes (By CEN) a start has been made to develop greater uniformity in the design of concrete by providing the basis of calculations and safety-philosophy. To disseminate new technologies and exchange practical experiences and knowledge in the field of concrete construction, a consensus about standards and design aids is a necessity. Based on the conclusions of an inquiry, recommendations are given how to inform the building practice on the new European Concrete Standards, and what is needed to gain support. Design aids as flow charts, graphs and tables and calculation examples on EC2 are being developed in cooperation, to be applicable in all design practices. Problems in the application of the EC2 in The Netherlands (which did occur in a special demonstration project on EC2 and ENV 206) and in general do occur, but future outlooks on the use of EC2 and design aids show a slow but inevitable progress.

Keywords: Eurocode 2, NAD, design aids, design process

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INTRODUCTION

Concrete construction has in all European countries a long tradition. After about fifty years of harmonization on a voluntary basis individual national design rules lead to results which differ more or less significantly. This shows the long breath we have to have to finish the process of European harmonization, which is in itself not very amazing:

also when national Codes are to be replaced by more sophisticated rules, as has happened in The Netherlands in 1993, the proces of adaption requires years. To meet the implementation problems, CEN has introduced a gradual approach by the use of prestandards, Principles, application rules and boxed values. The items which are now most discussed with regards to Eurocode 2 part 1, are its clearness and usefriendlyness, the reliability level and economical consequences in relation to national practice.

As the conversion from ENV to EN has been agreed lately, these problems should be solved before 1998.

DESIGN AIDS

The use of design aids in the design proces relating to Codes are common use in different European countries. Generally, the process is identified with Code Checking. Codes are presented in such a way that the design process is to be passed through in an easy way. Codes are not educational books: they are structured in a way to really design a structure, which means finding the right shape, size and material specifications for a structural solution. Although, during this preliminary design, anticipation on final code checking takes place.

Same can be said with reference to educational books: the are not ideal to perform a design job. The design process needs its own tools, and in each stage different tools are used, as shown in the table.

Tools freq. used	**Stage of the design process**
Reports (spec, sit.) Literature on spec. subjects Graphs and tables	1. Searching for shape (establishment of prismatic/haunched beams, mushroom /flat slabs)
Literature on costs	2. Sizing of given shape
Product information Computer progr. for optimization	3. Re-considering shape and sizes (considering possible mix design, fully/partially reinforced concrete, alternatives etc.)
Graphs and tables	4. Specifying reinforcement
Computer progr.	5. Code checking
Examples, tables	6. Detailing

Table 1. design tools used in different stages of the design process

This was shown by the inquiry in the design processes of different countries, performed within the EU-SPRINT programme by the Concrete Societies of Netherlands, Germany and UK. The use of present-day and future design aids was subject of a questionnaire. It showed a clear preference in all countries for conventional design aids as graphs and tables, with handbooks and calculation examples. The bureaus and engineers which were asked to fill in the questionnaire is not a mathemathical random sample: to reduce costs and work a selection of representative companies was made to achive representative figures. The results of the questionnaires are summarized:

1. Num. Examples
2. Charts and tables
3. Flow charts
4. Manual/Handbook
5. Computer programms

Some highlights from the questionnaire in The Netherlands (1994):

- Most of the leading engineers (74%) are informed roughly on EC2, so is their staff (58%). More than one half of the participants in one way or another has already studied on EC2 items, but 74% has never made any calculations.
- 81% has no company-initiated information flow on EC2 in their organization structure.
- The software used for different calculations is not prepared for EC2, according to 85%
- Estimating the costs for implementation of EC2, most engineers are concerned and think the costs overall will be reasonable/ to be expected (50%) or very high (47%). Especially computer programs and CAD systems are mentioned.
- Engineers are not unanimous in the judgement on possibilities concerning apllication of EC2:61 % sees no advantage in the use of EC2 with reference to freedom of design, against 32% who does see advantages.
- Examples, courses and design charts and tables are important for the use of EC2

Germany:

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- Examples, courses and design charts and tables are important for the use of EC2

United Kingdom:

- 75% of the participants have the opinion a Manual/handbook (with examples) is essential, followed by design charts and tables (42%). No immediate value is credit to Expert systems, CAD systems, textbooks and computer programms
- A simplified version of the Code is also mentioned, probably because this suits in the present design process in the UK
- A question about a realistic price for desing tools, showed that especially flow charts and a textbook have to be rather cheap.

However in the inquiry the use of IT tools are not envisaged as of main importance, IT will influence the design process more and more. The actual advance of the computer in the design process started in the seventies, although it has not -up untill now- pushed aside the tables and graphs yet. EC2 may be used as a starting point and may stimulate the development and implementation of advanced IT design aids by leading companies. The inquiry within this project shows that the ‘average’ engineer needs recognizable and applicable, often traditional design aids first, to get used to a new Code. The computer related design aids should be focussed on small, reliable, certified programms which are easy to applicate in bigger programms, according to some questionnaires. Also, the stage of the design process in which they are most helpfull is establishing excact sizes, specifying reinforcement and code checking. IT tools for optimization in a preliminary stage and for detailing or for the total design process are of second order.

Not to forget with regards to EC 2 is the difference calculating with EC 2 and calculating with the National Code for which most tools are developed.

Time is often the main criterium for choice of a design aid, and this will be more inportant in the future. For some stages of the process charts and tables are very well suited, for others computer programms are, depending on the preference of the engineer. Besides, not all items in the code are suited to be applied in computer programs.

One should also be aware of the difference between design tools for practical application (EC 2 experienced engineer) and tools for education (students, introduction of EC2)

EXAMPLE: USE OF EC2 IN THE NETHERLANDS

Untill now in The Netherlands not much experience has been gained on the use of Eurocode 2, although a persistant group of about hundred engineers and designers are working with or their work is related to the Eurocode. Last few years some companies are really working with EC2 e.g. because it was prescribed. In a large ‘Demonstration project’ on Eurocode 2 the Concrete Society together with experts from the building industry have designed a complete office building according to EC2, and have compared the results to a design concept with the national Code.

When the NAD was composed, deliberately a limited amount of adaptions on EC2 has foreseen, to anticipate on future harmonization.

Depending on the expectations one could draw specific conclusions with reference to the results. As there are differences, there are also many similarities. Main differences did occur on following items:

- Stress-strain diagram
- material properties
- strength classes
- concrete cover related to certain parts and crack width
- shear
- columns
- fire resistance
- bending

The required reinforcement for beams and plates according to EC2 is appr. 10% higher compared to the national Code. Main factor is the increased Load-factor for permanent loads. When the NAD is used this percentage is reduced because regular load factors from the Dutch Code are used.

For columns according to EC2 about 5–30% more reinforcement is needed. Again, using the NAD this is reduced to 0–20%. The difference increases when the influence of the normal force is getting bigger (with a materials factor).

With reference to deflection of slabs in EC2 some tables are provided. Basic ratios for span/effective depth are given related to the reinforcement ratio. Looking at the Dutch Code, using slabs with high permanent loads the basic ratio of span/effective depth a smaller ratio is allowed. Slabs with limited loads are allowed to have a higher basic ratio.

Eurocode 2 leads to about 10% higher reinforcement for shear. Applying the NAD about 5% difference remains. Sometimes this difference may be larger; e.g. when detailing provisions are taken into account. In Eurocode 2 minimum shear reinforcement is at least two times the value in the Dutch Code.

The size (thickness) of flat slabs is -related to punching- about 30% higher in Eurocode 2. at a much earlier stage column heads or extra reinforcement for punching is needed. As in The Netherlands this kind of slab is frequently used, this would not be appreciated very much.

Furthermore, the aspect punching is subject to a major difference between EC2 and the Dutch Code (value for 1). Another major difference is the way excentric punching is handled; a rather simple method in used in EC2, but the question can be raised if this method follows the real punching behaviour.

The approach for torsion in EC2 is based on the fact that all moments need reinforcement, whereas the Dutch Code provides a threshold. These different approaches may lead to different amounts of reinforcement, which is fostered by the higher minimum value for reinforcement stirrups. Furthermore, detailing provisions in EC2 may lead in general to more required reinforcement ratios.

Again, the NAD reduces these differences.

Some of the results are shown in figures 1–3.

Summarized, some calculations showed a more conservative approach (related to reinforcement %) with EC2. On the contrary, e.g. shear calculation methods used in the Dutch Code are far more concervative compared to EC2.

Use of EC2+NAD appeared not to be not complicated with reference to the National Dutch Code. Structure of the Eurocode 2 is nevertheless more complicated (where is what to be found). Sometimes it is hard to get a clear view on all relevant aspects and articles related to a specific subject.

A ’representative’ designer was able to use EC2+NAD in a few days, and had the opinion that design aids as graphs, tables and flow charts were quite necessary. As they are to be supplied by the SPRINT project in March 1996, and the official NAD and ENV 1992–1–1 have been published in The Netherlands last year, nothing is keeping the Dutch engineer from use of EC2+NAD, however the acceptance of calculations by the building authorities should be clear. At the moment the Building Authorities are not all quite convinced of the equality of the design concept of EC2+NAD and the Dutch Code, and they have no experience in control of EC2 calculations. Administrative and organizational initiatives are

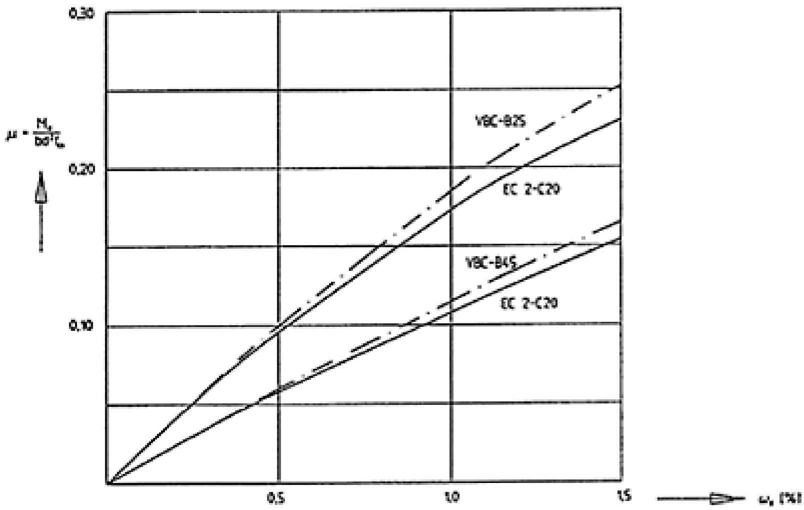

Figure 1. Bending without longitudinal force; differences in reinforcement ratio related to μ=M_d/bd^2f_{ck}

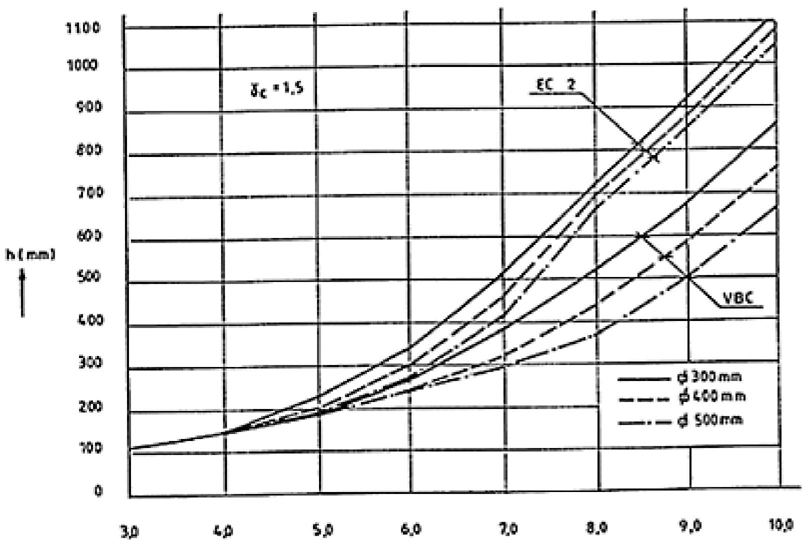

Figure 2. Thickness of slab (h) related to span (I) for EC2 and Dutch Code (safety factor 1, 5) → ℓ (m)

taken at the moment to improve the situation. One can deliver EC2+NAD design calculations at the desk of the Building Authority, but still has to prove the conformity with the National Dutch Code before they are accepted. It is to be foreseen that in 1996 in The Netherlands more and more EC2+NAD calculations will be made and authorized.

The Concrete Society of the Netherlands is stimulating this process by offering the building practice a ’package for the use of EC2’, containing a (draft) translation of ENV 1992–1–1, the Book on Design Aids, an informative brochure on use and implications and the official issue of the NAD.

ENV 206

With reference to ENV 206 this document was also ’tested’ the process of execution of concrete structures at the building site of two tunnels at Schiphol Airport. It has to be emphasized that the application of ENV 206 was not initiated-to adapt the ENV 206 to the relevant Dutch Code (VBT/VBU). Our opinion is that a widely accepted Code has to contain performance requirements in order to enable all countries to use their national methods as long as they satisfy these requirements. Although there are textual remarks (more concrete), too many references and too many notes, the ENV 206 is generally considered as applicable, apart from the folowing aspects:

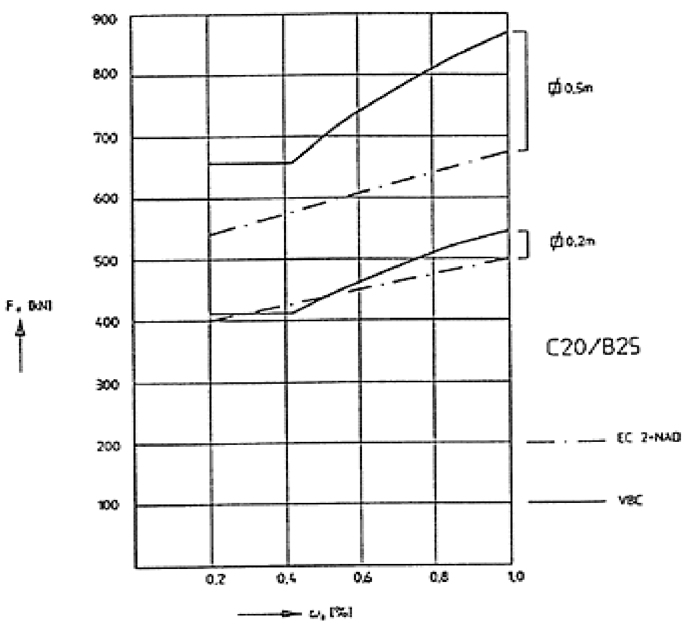

Figure 3. Value of Punching Load F_d in relation with reinforcement ratio (h_s=0.3 m., column 0.2–0.5 m.)

– adaption of chapter 11 for the benefit of a clearer structure
– a choice should be made on the kinds of cements allowed for application
– more sophisticated methods for measurement and determination should be mentioned (e.g. 7.2.4 drying methods and 10.6.3. maturity concept)

As The Netherlands has a different Code for ’Performance, production and compliance criteria’ (VBT) and ’Prescriptions for execution of concrete structures’ (VBU) adaption of one ENV for both different Standards will be very difficult, and initiatives are undertaken to compose a European Standard on Execution on a European level, which has been worked out in a Working Group on the subject.

Especially the ENV 206 has a clear ‘visual’ impact on the execution of concrete structures in different countries, and is therefore strongly prescribed to performance criteria. It is inevitable that adaptions in the production and execution process of concrete are needed, but they can be minimized. Untill agreement is achieved in the CEN Committees on different subjects of the ENV 206, the could be removed from the text (preliminarily). For it is clear that gradual implementation, parallel to the National Codes, is the most convenient solution.

FUTURE OUTLOOKS

EU with an approved Eurocode Programme means a market with more ideas and funds to initiate, develop and implement new technologies. As EC2 provides new challenges for automatic procedures because it is not related to any specific (conservative) design practice, this is in favour of market possibilities for development of software. Especially for smaal countries certain developments are just unfeasable when they are taken up on a national basis only. EC2 is in fact more intricate in the procedures to follow compared to procedures from the past that are less formal but easier to apply. IT can and will probably play a major role in handling of partial safety coefficients and the increased amount of checks on load combinations and limited states to be envisaged.

The use of the NAD is an inevitable step towards total implementation of the Eurocodes. Preferably, NAD’s should be as short as possible, to facilitate introduction of the Eurocodes as EN’s. When the national NAD is well structured and concise, principally it is not a problem to use it in practice, as trial calculations in The Netherlands point out.

With the increasing activity on IT tools, the use of CAD/CAM is increasing also in many countries. Especially this area needs a harmonized Code, for the different systems, various national Codes, symbology, language and representation of reinforcement are still very far from each other. E.g. an ’English’ reinforcement drawing may well be totally unusable in Germany. For the execution of concrete structures and information exchange in the European building process these aspects are of main importance, beside the design Code.

New developments, leading to better hardware and software for less costs will influence the future of design. As use of commonly used tools as graphs and tables, IT tools should be incorporated in education. International cooperation to develop and implement such software, based on EC2, should be made. When we look at the European steel construction initiatives in this field, we may conclude they are ahead of the concrete IT-design and concrete IT education programmes (of course Standards for Steel Construction are more suitable for IT application). The availability of flow-charts for EC2 is a step forward in this matter. Flow charts are very useful for development of software programmes.

Government and Industry should not hesitate to use advanced but realistic information systems in construction. But together with implementation of these developments, quality control and the 'engineers touch' will be needed. The engineer is responsable for its design and the calculations; he cannot blame the IT tools he has used.

Further to EC2, in 1998 a package of (a limited number) EN's for design of concrete structures is foreseen. When we look at the publication of European technical books on concrete construction, we may determine a small but clear increase of books on Eurocodes.

With this in mind, and knowing that some aspects of Eurocode 2 need a sound discussion of some design problems, the time-path towards 1998 is short, however achievable. With proper design aids and increasing use of IT tools in the near future, the treshold to use EC2 is lowered.

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IMPLICATIONS OF STRUCTURAL EUROCODES

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ABSTRACT. The paper reviews the process of preparing the structural Eurocodes over the past two decades. The structural Eurocodes form an important link in the proposed plan for removal of barriers to the trade among the Member States of the European Community. The process of preparing the Eurocodes, however, has been slow and with difficulties which could be expected when it is necessary to have consensus among various countries on a large number of issues; for example, presentation of design rules, the extent of guidance given by codes of practice, input of research into the codes, the status of codes of practice, etc. In spite of some such problems, the drafting of Eurocodes has made considerable progress and it is important to examine their implications for the construction industry.

Keywords: European Community, Eurocodes, European Product Standards, Structural design rules, National Standards.

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INTRODUCTION

The European Community (EC) had the concept of creating a common market, which was in a continuous state of development since the year 1950. In 1985, therefore, the Member States asked the Commission of the European Communities (CEC), which is one of the institutions of the EC, to put forward a plan to achieve a unified internal market by the year 1992. It was recognised that the construction industry and construction products represented a large proportion of the market. For a considerably long time, the EC had tried to introduce an agreed EC standard for health and safety, with adjustments to the individual national regulations. The CEC recognised that this task would be too difficult and it would not be practicable to achieve it within a finite time-scale. The CEC adopted a "new approach" to technical harmonisation of standards of health and safety for the EC. The EC policy on technical harmonisation is put in place through its Directives. For the construction industry, the most important Directive is the Construction Products Directive (CPD).

The CPD is aimed at providing freedom of sale and use of construction products, provided that the products have such characteristics that the structures in which they are incorporated meet the following essential requirements.

1. Mechanical resistance and stability
2. Safety in case of fire
3. Hygiene, health and environment
4. Safety in use
5. Protection against noise
6. Energy conservation and heat retention

Each requirement is associated with an "Interpretative Document (ID)", which interprets the expectations of the requirement. In the main, the ID's are not intended for the use of designers of structures. The ID's are often described as "the codes for writers of the Eurocodes". For example, the "Limit State Concept" is incorporated in the ID for the requirement "Mechanical resistance and stability" and this concept is the basis of all structural Eurocodes.

The structural Eurocodes are intended to serve as reference documents for two main purposes. First, they should serve as a means of proving compliance of building and civil engineering works, mainly with the essential requirement "Mechanical resistance and stability". The Eurocodes also concern the second requirement "Safety in case of fire", since fire is accounted for as an accidental action in the design of structures. Second, they are expected to form a framework for drawing up harmonised technical specifications for construction products.

The CEC had intended that the Eurocodes should initially serve as an alternative to the different national codes in use in the various Member States. The CEC had expected that the Eurocodes would ultimately replace these national codes. In 1990, after consulting the Member States, the CEC transferred the work of further development, issue and updating of the structural Eurocodes to CEN, the European Committee for Standardisation. CEN technical committee CEN/TC250 has the responsibility for these structural Eurocodes. The sub-committees of CEN TC 250 (SC1, SC2 etc) are responsible for Eurocodes 1, 2, etc.

CURRENT POSITION

In the first instance, various parts of the Eurocodes are published as ENV (European pre-standard), when the final drafts prepared by the project teams are approved by the appropriate sub-committee of CEN/TC250. The Member States are expected to encourage the use of ENV stage codes accompanied by the National Application Documents (NAD's), which contain the following information:

a) The values of parameters acceptable to the National Body (for example, BSI in the UK) in place of the "Boxed" values given in the ENV; (The boxed values are without prejudice and are not necessarily those which will be agreed by the UK at the EN stage.)
b) Reference to supporting standards and codes which are necessary for the use of the ENV. (e.g., loading/actions, material specifications, fire etc.);
c) Additional information and guidance required for the use of the ENV.

In the UK, the ENV, NAD and the National Foreword are published by the British Standards Institution (BSI) as single document "DD ENV .." as a "draft for development". Preparation of NAD's is overseen by the relevant sub-committee of the BSI. The actual work is usually supported from the Department of Environment (DOE) and it is led by the Building Research Establishment (BRE), on the basis of the following examinations and studies:

i) Textual Examination

A detailed examination is carried out to provide comments on the following topics, using the relevant national standard as a model:

a) General lay out, cross-referencing and scope of the subjects covered by the ENV;
b) Consistency of the draft rules;
c) Significant differences in formulae and procedures;
d) General clarity, readability and usability of the draft;
e) Implications for designers' legal responsibilities;
f) Feasibility for outlining a step by step design procedure.

ii) Calibration Studies

These studies involve comparisons of capacities of structural elements derived according to the existing British Standards with those derived according to the relevant Eurocodes. The draft Eurocodes contain numerical values identified by enclosing numbers in "boxes", which are given by the drafters as "indicative" values. Member states are permitted to specify alternative values, which could be derived as a result of satisfactory conclusions of these studies. The indicative values in the drafts are replaced by national boxed values, so as to achieve the following objectives:

i) the use of the boxed values does not permit construction of markedly less reliability than the least acceptable in current practice;
ii) the cost of construction in real terms is not greater than what is being usually incurred at present.

The following list gives a plan for the main parts of Eurocodes related to buildings, based on the current information. This list gives the ENV number of the approved and future parts, and the title and number identifying the part and sub-part. The sub-part numbers indicated as -x, -y, and -z will be given in chronological order of approval. The last column indicates whether the European Pre-standard is in English (E), French (F) or German (D), or whether it is approved and it is in the process of publication (Ap), or at which target date (9Y-MM) the approval is foreseen. A letter for the language (E,F or D) followed by a date YY-MM indicates the month in which the publication is foreseen in that language.

Table 1: Eurocode EC1 (Basis of Design and Actions on Structures)

ENV 1991–1:1994	Part 1: Basis of design	E
ENV 1991–2– 1:1995	Part 2–1: Actions on structures Densities, self- weight and imposed loads	E 95–02
ENV 1991–2– 2:1995	Part 2–2: Actions on structures Actions on structures exposed to fire	E 95–02
ENV 1991–2– 3:1995	Part 2–3: Actions on structures Snow loads	E 95–02
ENV 1991–2–4	Part 2–4: Actions on structures Wind actions	E 95–03

Table 2: Eurocode EC2 (Design of concrete structures)

ENV 1992–1– 1:1991	Part 1–1: General rules and rules for buildings	E, F, D
ENV 1992–1–2	Part 1–2: Structural fire design	Ap
ENV 1992–1– 3:1994	Part 1–3: Precast concrete elements and structures	E
ENV 1992–1– 4:1994	Part 1–4: Structural lightweight aggregate concrete	E
ENV 1992–1– 5:1994	Part 1–5: Un-bonded and external tendons in buildings	E
ENV 1992–1– 6:1994	Part 1–6: Plain concrete structures	E

Table 3: Eurocode EC3 (Design of steel structures)

ENV 1993–1–1:1992	Part 1–1: General rules and rules for buildings	E, F, D
ENV 1993–1–1:1992 /A1:1994	Part 1–1/A1: General rules and rules for buildings (Annexes D and K-revised)	E
(ENV 1993–1–1/A2)	Part 1–1/A2: General rules and rules for buildings (Annexes G, H, J-revised, N and Z)	95–05
ENV 1993–1–2	Part 1–2: Structural fire design	Ap
ENV 1993–1–3	Part 1–3: Cold formed thin-gauge members and sheeting	Ap

ENV 1993–1–4	Part 1–4: The use of stainless steels	Ap

Table 4: Eurocode EC4 (Design of composite steel and concrete structures)

ENV 1994–1– 1:1992	Part 1–1: General rules and rules for buildings	E, F, D
ENV 1994–1– 2:1994	Part 1–2: Structural fire design	E

Table 5: Eurocode EC5 (Design of timber structures)

ENV 1995–1– 1:1993	Part 1–1: General rules and rules for buildings	E, F, D
ENV 1995–1– 2:1994	Part 1–2: Structural fire design	E

Table 6: Eurocode EC6 (Design of masonry structures)

ENV 1996–1–1	Part 1–1: General rules—Rules for reinforced and unreinforced masonry	Ap
ENV 1996–1–2	Part 1–2: General rules—Structural fire design	Ap
(ENV 1996–1–3)	Part 1–3: General rules—Detailed rules on lateral loading	96–04
(ENV 1996–2)	Part 2: Special design aspects	96–10

Table 7: Eurocode EC7 (Geotechnical design)

ENV 1997–1:1994	Part 1: General rules	E
(ENV 1997–2)	Part 2 :Geotechnical design assisted by laboratory testing	96–10
(ENV 1997–3)	Part 3 : Geotechnical design assisted by field testing	96–10
(postponed)	(Part 4: Specific geotechnical structures)	

Table 8: Eurocode EC8 (Design for earthquake resistance of structures)

ENV 1998–1– 1:1994	Part 1–1: Seismic actions and general requirements for structures	E
ENV 1998–1– 2:1994	Part 1–2: General rules for buildings	E
ENV 1998–1– 3:1995	Part 1–3: Specific rules for various materials and elements	E 95–02
(ENV 1998–1–4)	Part 1–4: General rules—Strengthening and repair	95–05

Table 9: Eurocode EC9 (Design of Aluminium Alloy Structures)

(ENV 1999–1–1)	Part 1–1: General rules—General rules and rules for buildings	97–10
(ENV 1999–1–2)	Part 1–2: General rules—Structural fire design	97–04
(ENV 1999–2)	Part 2: rules for structures susceptible to fatigue	97–04

After the ENV stage of some 3 years, the ENV's should be transformed into EN (European Standard). All the available ENV's can be purchased from the European National Standards Institutes which are members of CEN. For countries which are not members of CEN, the English version can be purchased from BSI in London, the German version from DIN (Deutsches Institut für Normung) in Berlin, and the French version from AFNOR (Association Franchise de Normalisation) in Paris.

SUMMARY OF UK ACTIONS

The UK experts have had a large share of representation on the project Teams drafting the Eurocodes. In addition, other activities dedicated to the Eurocode cause have been progressing, mainly sponsored by the DOE, with participation by the Building Research Establishment (BRE), BSI and a number of private sector experts. Some of these activities are listed below.

Development of National comments

At every stage of the drafting of codes, the Building Regulations Division of the DOE has been responsible for collecting the national comments and submitting them to the CEC earlier and now to the CEN, in collaboration with the appropriate BSI committee. In 1994, the DOE sponsored a "parallel design exercise", based on the design of the future DOE Headquarters building in London and using ENV Eurocodes EC2 and EC3. This exercise has contributed to the UK national comments on these Eurocodes.

Technical Coordinators

Technical Coordinators have been appointed for each Eurocode, through the commissions funded by the DOE, to serve the relevant BSI committees. Each Technical Coordinator represents the views of the BSI committee at the CEN subcommittee meeting and reports to the chairman of the BSI sub-committee, and liaises with the DOE on matters concerning implications for Building Regulations.

Other measures for promoting trial use at ENV stage

The DOE has drawn public attention to the publication of ENV's, in a circular concerned with implementation of the Construction Products Directive (CPD). This circular referred to a separate note describing the documents produced by private organisations, such as worked examples, simplified versions of the Eurocodes, comparative studies of national and European codes etc. At the same time, the DD ENV 1992–1–1 and DD ENV 1993–1–1 have been mentioned in the Approved Document (Part A-Structure) as suitable for compliance with the requirements of Schedule 1 of the Building Regulations 1991, applicable in England and Wales. These references are repeated in the National Forewords to all individual DD ENV's.

Concise Eurocode and Worked Examples

In 1993, the British Cement Association published a Concise Eurocode for the design of concrete buildings to enable a designer to change from BS8110 to Eurocode 2 with the minimum of effort. This document contains all the information needed for the routine design of normal building structures.

The British Cement Association (BCA) has also published a 256 page book of worked examples for the design of concrete buildings. This major contribution to the promotion and use of Eurocode 2 has been funded by BCA and DOE. The book incorporates the combined efforts of design engineers from BCA, Ove Arup & Partners and S B Tietz & Partners.

Similar work has been done by the Steel Construction Institute and other trade organisations, for assisting the use of other Eurocodes.

MAJOR ISSUES CONCERNING THE EMERGING EUROCODES

Order of production

The ENV's of structural material codes (EC2, EC3 for concrete, steel etc.) were issued in advance of the ENV versions of EC1 (Basis of Design and Actions). The NAD's of EC2 and EC3 were based on their comparisons with the corresponding national codes, using the information given in the BS loading codes. As a result, the NAD for ENV Eurocode EC1 may have to align the values of loading closely with the corresponding figures in the BS, in order to maintain the validity of the comparison exercises used as the basis of NAD's of EC2, EC3, etc.

Compatibility between European product standards and the Eurocodes

The European standards are issued as pr-EN drafts and they are expected to become EN's within a short period. This could have implications for the compatibility between the product standards and the Eurocodes; for example, the precast concrete products and Eurocode EC2. Generally, the product standards are supposed to be based on the structural Eurocodes, but the Eurocodes are far from their final EN stage. It is understood that this question is being addressed by a Group including the Chairman of TC/250 and the other TC's who are responsible for individual product standards. This question is also relevant to the Eurocode Parts 1.2 (Structural Fire Design), which refer to the standards on certain protection measures and tests.

Nature of the presentation

The Eurocodes are presented in a manner different to the BS codes. Some of the problems for UK engineers arise due to the perception of the codes of practice related to the issues of professional liabilities. Also, the Eurocodes seem to contain a large part of text which is not considered useful by the UK engineers and they often have a difficulty in coming to the part which they consider as essential. Additionally, some of the rules in the Eurocode parts, fire parts in particular, are based on research which has not been tried and tested to the standards expected of research in the UK.

FUTURE OF THE EUROCODES

At their recent meeting in Lisbon, the committee stated key target dates, relating to those parts of the Eurocodes necessary for the design of buildings. CEN/TC250 is developing an action plan to achieve the following:

i) All the EN Eurocodes necessary for the design of structures for buildings comprising Basis of Design, Loading and General Design Rules including those for Fire and Seismic Resistance, to be published by 1 January 2000.
ii) The first group of documents consists of Part 1 and Parts 2–1, 2–3 and 2–4 of Eurocode 1 and Parts 1–1 of Eurocodes 2, 3 and 4 to be published before the end of 1998.

The ENV's for bridge parts are to be approved by the end of 1996 to allow their conversion to EN's before the end of 2001.

The development of the Eurocodes is still a "step-by-step" process. The plans for action and the dates, therefore, can only be conditional or subject to completion of certain key activities. CEN rules require that, when a full EN standard is issued, the conflicting national standards have to be withdrawn within 6 months. However, in the case of Eurocodes, the CEC mandate for CEN specifically states that this will not happen. After the Eurocodes reach EN status, CEN, CEC and the national governments will consider the arrangements for superseding national standards and they may agree a period of coexistence of the national standards and the Eurocodes.

The regulatory systems of other countries differ from the UK system. In some countries, the adoption of structural Eurocodes may produce legal and administrative problems that may be more difficult to overcome, quite apart from any commercial or technical issues. It will be some time before one can say anything definite about the decision for replacing the national standards by Eurocodes or the length of the transition period. The DOE is actively pursuing these issues with the CEC.

The best indication so far on transition, is that several countries are already revising their national standards for steelwork to bring them voluntarily closer to the Eurocode 3 (for example The Netherlands and Switzerland). In France the ENV version of EC3 already has full legal equivalence with other standards. The smaller countries (Ireland, Luxembourg etc) are amongst the keenest on harmonisation. Even CEN associates (for example Rumania) are issuing versions of EC3 as national standards.

The CEN asked the 18 members, if they should proceed to convert the present ENV version of EC2, EC3 and EC4 into EN's and, if so, whether with or without technical changes. This enquiry has received a positive response. The CEC/CEN agreement provides that, in any case, CEN cannot proceed with the conversion without the consent of the European Union and the European Free Trading Area. It is expected that CEC will base their decision on views from national governments through the Standing Committee for the Construction Products Directive.

In accordance with CEN rules, all standards are subject to a 5 year review and the Structural Eurocodes will be no exception. The plans for final versions in 1998 will, therefore, require a rapid conversion. (There is little more than 12 months for technical discussion, the rest is needed for translation, editing and such other procedures). The timetable can only be met if technical changes are restricted. It is envisaged that delegations will agree to this, only if they are promised an early start on a more radical review in some 5 years time. The CEN rules also allow for a maximum 3 amendments between revisions. Whether this will be found useful (or even practicable) remains to be seen.

The Eurocodes provide a relevant method for demonstrating suitability of products. Although the Eurocodes are not formally linked with the CPD, they have a role in the practical implementation of the CPD. In addition, the Eurocodes have a role concerning responsibilities on designers, amongst others, which are stated in the new Construction (Design and Management) Regulations 1995.

The Eurocodes have an important role in relation to Public Procurement (works, utilities). Public Procurement Directives require tenders to use European standards, where they exist. A number of CEN product standards (typically for fabricated products, rather than basic products) have a need for design rules. The policy is that they should refer to Eurocode rules, as far as practicable. This is an important reason why the EN versions of Eurocodes are needed promptly.

FRACTURE MECHANICS PARAMETERS OF CONCRETE: TEST METHODS DEVELOPMENT AND HARMONIZATION OF STANDARDS

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ABSTRACT. The study, which forms part of a research programme investigating the fracture mechanics of concrete, deals with the effects of coarse aggregate (CA) volume concentration on crack initiation and failure of concrete. The test mixes were designed with ordinary Portland cement and 0, 50, 75 % CA replacement levels and a constant workability of 70 mm slump. Fracture mechanics parameter values for different types of concrete are determined by stable three-point bend tests on notched beams. On the Republic Institute of the Innovative Technologies Byelorussian State Polytechnic Academy goal- oriented research has been made for some recent years in the direction of creation of concrete classification by crack resistance based on the force and energetic criteria of fracture mechanics. The complex experimental researches of the influence of type, activity, rate of cement; water/cement ratio; existence of chemical admixtures on concrete resistance to the crack formation and development have been carried out [5–7]. All calculations are carried out on three- point bend beams with a notch. Applying special testing devices with high rigidity and quick feed-back in the loading mechanism there is a chance to conduct stable three-point bend tests on notched beams. The classification of concrete crack resistance on the base of force and energetic of fracture criteria is proposed.

Keywords: Heavy—Weight concrete, Crack resistance, Strength, Fracture mechanics, Force and energetic parameters, Coarse and fine aggregates, Classification.

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INTRODUCTION

Situations where tensile strength and tensile toughness are of particular importance are, for example, anchorage of deformed bars, shear forces in slabs and beams, splitting under concentrated forces, and unreinforced pipes.

With authors participating State Standard 29167–91 “Concretes. Methods for determination of fracture toughness characteristics” has been developed.

This paper examines how the crack resistance of concrete varies with the volume concentration of different aggregates .

By this time the large volume of experimental data has been accumulated on the main parameters of fracture mechanics of concrete [1–7]. Nevertheless,, the application of gained values of force and energetic parameters fracture parameters in practical calculations is connected with many difficulties. It is explained by the lack of normalized fracture parameters gained by completely balanced diagrams of deformation.

The research has been started in the CEB in the direction of development of concrete classification approach. The concrete classes statements defined by the parameter of fracture energy Gf, applied for the fracture were included in the cods model. Concrete are classified on dependence on the size of coarse aggregate [8] :

$$G_f = \alpha_f \frac{f_{cm}}{f_{cmo}},$$

where f_{cmo}=10 N/mm^2; $f_{cm}=f_{ck}+D_f$; D_f=8 N/mm^2; f_{ck}—concrete cylinders compressive strength, N/mm^2. The coefficient αf depends on the maximum aggregate size d_{max}.

The CEB approach to concrete has some defects. Firstly, the classification of concrete by crack resistance using only one parameter—G_f (even if it is very essential), is not complete. Fracture energy G_f does not give the differential estimation on the cracking (on the pre-peak stress stage) and the growth of crack in concrete (on the post-peak stress stage). In dependence on the type of stressdeformed state of a concrete structure and conditions of its exploitation this may be of dominant value during the comparison of different concrete contents.

Secondly, the main influencing factor-the size of coarse aggregate d_{max} seems to be insufficiently based. More correctly from our point of view is to use the index of aggregate surface. As the fracture of heavy-weight concrete occurs in the contact zone with coarse aggregate, this characteristic defines concrete crack resistance. Equal value to this parameters may be content by volume of coarse aggregate, as it is easily defined and more clear to a production worker. Introduction into the volume concentration of coarse aggregate analysis supposes consideration of property matrix-cement-paste. Besides the length of progress main crack defined by volume concentration and grading of coarse aggregate, concrete crack resistance will be influenced also by the strength and crack resistance of cement-paste; we can say this individually resists to progressing of main crack in a heavy-weight concrete.

In the third, the classification given by EuroCode does not lightweight porous aggregate concretes, the wide application of which is limited because of insufficient knowledge of their crack resistance [5].

DETERMINATION OF FORCE AND ENERGETIC PARAMETERS OF CONCRETE

Specimens—prisms measuring 100×100×400 mm of each type of concrete were prepared in steel moulds and stored for 180 days. The notches (35 and 5 mm) were cut using a diamond saw. The width of the notch was 2 mm.

In applying special testing devices with high rigidity and quick feed-back in the loading mechanism there is a chance to do stable three—point bend tests on notched beams (Figure 1).

The three-point stable diagrams were used for determining energetic and force parameters *(G_i, G_f, J_i, K_{ic})*, where G_i—the energy release rate; G_f—the fracture energy is defined as the amount of energy required to develop a crack of one unit of area, and in other words, the critical strain energy release rate or toughness; K_{ic}—the critical stress intensity factor; J_i—static *J*-integral.

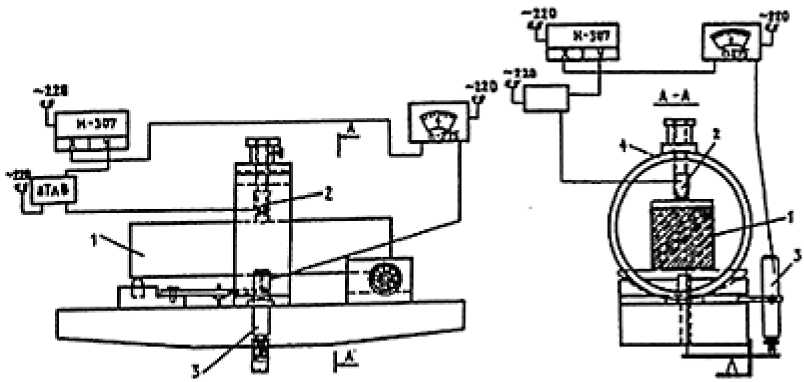

Figure 1 The scheme of tasting machine for three-point bend test

1—specimen-notched beam; 2—apparatus recoding the load;

3—apparatus recoding the deflections; 4—rigid ring element.

RESULTS AND DISCUSSIONS

The stable load—deflection curves from three-point bend tests for different types of concrete are presented in Fig. 2, 3. As can be seen in Fig. 4, 5 and Table 1, the strength (R), deformativity (Eb), crack resistance of concrete are strongly affected by the quality of the aggregate. In comparison with cementsand mortar (j=0), the compressive strength of granitoid concrete with volume concentration of crushed-stone j=0.353 was increased by 6.5% and with j= 0.621—by 55% respectively. The compressive strength of gravel concrete decreases when the coarse aggregate /cement-sand mortar ratio increases: by 1.4% (j=0.375) and by 25% φ=0.643).

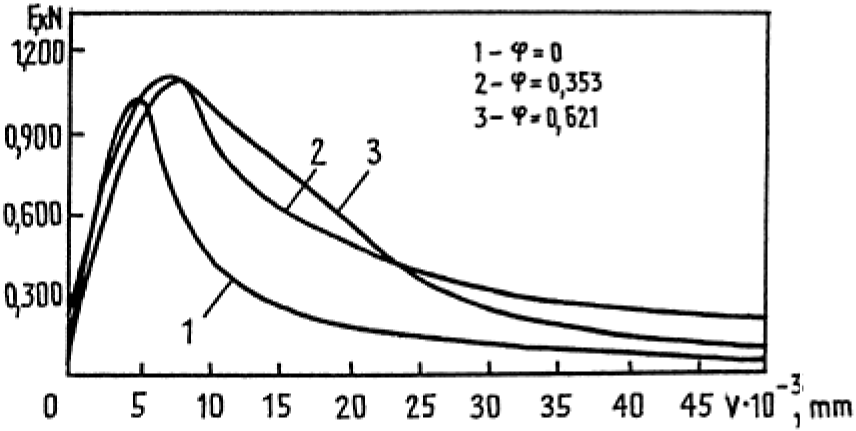

Figure 2 The load—deflection curves of crushed—stone concrete

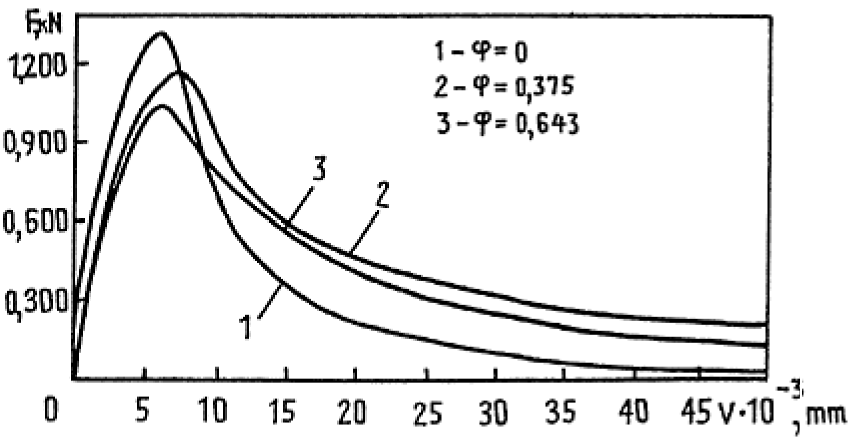

Figure 3 The load—deflection curves of gravel concrete

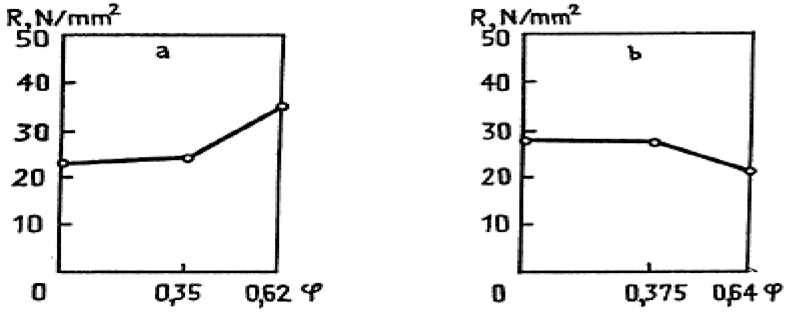

Figure 4 The influence of the type and concentration of the coarse aggregate on the strength of heavy-weight concrete

a) crushed—stone aggregate; b) gravel

In Fig. 5 the experimental energetic (G_f, J_i) and force (K_{ic}) parameters of concrete against volume concentration of granite or gravel are presented. It can be seen that both the resistance to crack initiation (in the ascending branch) and resistance to crack propagation (in the descending branch) increase with the increasing volume concentration of crushed stone approximately by 2 times (j=0.353), by 2.3 times (j=0.621) in comparison with the cement-sand mortar. Increase in resistance to growth of crack for concrete with the concentration of crushed stone (j=0.353) is essential (G_f by 79 %).

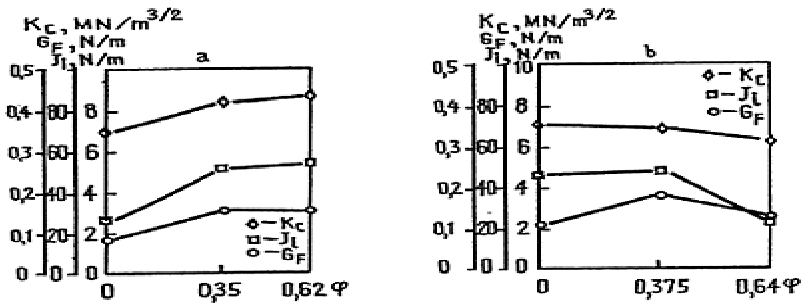

Figure 5 The influence of the type and concentration of the coarse aggregate on the crack resistance of heavy—weight concrete

a) crushed—stone aggregate; b) gravel

Table 1 The influence of the type and concentration of the coarse aggregate on the strength, deformation and crack resistance of heavy-weight concrete

TYPE OF COARSE AGGREGATE STUFF	VOLUME CONCEN TRATION OF CA	PARAMETERS OF STRENGTH AND CRACK RESISTANCE					
		R, N/mm^2	$Eb \cdot 10^{-4}$ N/mm^2	G_i, N/m	G_f, N/m	J_i, N/m	K_{ic}, $MNm^{-3/2}$
CS	0.0	23.0	2.674	5.25	17.56	2.63	0.38
	0.353	24.5	2.758	8.79	31.49	5.07	0.49
	0.621	35.7	3.205	9.16	30.77	5.38	0.54
G	0.0	28.5	2.938	7.09	20.88	4.59	0.46
	0.375	28.1	2.921	7.21	37.88	4.79	0.46
	0.643	21.2	2.573	4.61	27.75	2.43	0.32

CS—crushed stone aggregate; G—gravel

Coarse aggregate is more important for greater crack resistance than for strength of heavy weight concrete.

There were some differences between the results obtained with crushed stone concrete and those with gravel concrete.

For the gravel concrete with the moderate concentration of the coarse aggregate *j*=0. 375 there was a considerable increase in crack resistance parameters at the post-peak stage (G_f by 81%), and their constant values at the pre-peak stage of deformation (Figure 5).

For the gravel concrete with the maximum concentration in coarse aggregate *j*=0.643 there was the considerable decrease of the crack resistance parameters at the both stages of deformation.

PRACTICAL APPLICATION OF RESULTS

The results of this study show conclusively that the use of optimum volume concentration of coarse aggregate in concrete contributes significantly to its crack resistance.

The optimum volume concentration is less than maximum concentration. We suggested that the method of concrete classification by crack resistance should be built in this sequence:

a) normalized values of crack resistance parameters depending on the concrete classes by strength;

b) crack resistance parameters depending the strength of cement-paste and volume concentration of coarse aggregate;
c) normalizing of fine aggregate concrete crack resistance depending on cement stone property and volume concentration of fine aggregate.

Thus, from crack resistance of cement stone we shift to cement-sand mortar and concrete of different classes with normalizing of crack resistance.

These investigations made it possible to obtain expressions for determinations of force and energetic parameters of concrete fracture on crushed stone and gravel with the account of coarse aggregate concentration its strength of cement-paste:

G_i=0.8496+0.3951B,
G_f=17.177+0.9795B,
J_i=0.4125+0.2752B,
K_{ic}=0.082+0.01767B,

where B denote the specified characteristic compressive strength in N/mm (concrete grades for normal weight concrete).

Table 2 Crack resistance parameters of heavy-weight concrete

CRACK RESISTANCE PARAMETERS	CONCRETE COMPRESSIVE STRENGTH **B** (N/mm2)											
	B10	B12.5	B15	B20	B25	B30	B35	B40	B45	B50	B55	B60
G_i (N/m)	4.8	5.8	6.8	8.8	10.8	12.7	14.7	16.7	18.6	20.6	22.6	24.6
G_f (N/m)	27	29	32	37	42	47	51	56	61	66	71	76
J_i (N/m)	2.3	3.0	3.7	5.1	6.5	7.8	9.2	10.6	12	13.3	14.7	16
K_{ic} (MN/m$^{3/2}$)	0.3	0.37	0.41	0.5	0.59	0.67	0.76	0.84	0.93	1	1.1	1.19

In the tables 3, 4 are presented crack resistance indexes G_f, K_{ic} of concrete based on crushed stone and gravel in dependence on fine concrete strength and volume concentration of coarse aggregate.

Table 3 Fracture energy G_f(N/m) of Crushed-Stone Concrete

VOLUME CONCENTRATI ON OF COARSE AGGREGATE φ (%)	STRENGTH OF CEMENT-PASTE R_{cp} (N/mm^2)					
	15	25	37	50	60	70
10	28.9	36.5	45.7	55.6	63.2	70.9
20	29.2	36.8	46.0	55.9	63.5	71.2
30	29.5	37.2	46.3	56.2	63.8	71.5

40	34.3	42.0	51.1	61.0	68.7	76.3
50	35.8	43.4	52.5	62.5	70.1	77.7
60	37.2	44.8	54.0	63.9	71.5	79.4
70	38.6	46.2	55.4	65.3	72.9	80.6

Table 4 Critical stress intensity factor K_{ic} (MNm-$^{3/2}$) of gravel concrete

VOLUME CONCENTRATI ON OF COARSE AGGREGATE φ (%)	STRENGTH OF CEMENT-PASTE R_{cp} (N/mm^2)					
	15	25	37	50	60	70
10	0.288	0.425	0.590	0.769	0.907	1.045
20	0.286	0.424	0.588	0.768	0.905	1.043
30	0.285	0.423	0.587	0.767	0.904	1.042
40	0.232	0.369	0.534	0.713	0.851	0.988
50	0.217	0.355	0.519	0.699	0.837	0.971
60	0.203	0.341	0.505	0.685	0.823	0.960
70	0.189	0.327	0.491	0.671	0.809	0.946

By this time the large volume of experimental data has been accumulated on the main parameters of fracture mechanics of fine concrete [1–7]:

K_{ic}=m$_1 R_{bt}$,
K_{ic}=m$_2 R$,
K_{ic}=nK_{ic}’,

where R, R_{bt}—are compressive and tensile strength of fine concrete respectively (N/mm^2); K_{ic}—critical stress intensity factor of fine concrete (MN/mm$^{3/2}$); the coefficients m_1, m_2, n depend the volume concentration of fine aggregate (Table 5); K_{ic}’—critical stress intensity factor of cement stone (MNm$^{-3/2}$).

Table 5 Crack resistance of fine concrete

VOLUME CONCENTRATION OF COARSE AGGREGATE φ (%)	COEFFICIENTS		
	m_1	m_2	n
Less 10	0.25	0.006	1.00
10–20	0.20	0.007	1.10
20–30	0.15	0.008	1.15
30–40	0.14	0.009	1.10

40–50	0.13	0.010	1.05
50–60	0.12	0.010	0.85
More 60	0.11	0.010	0.50

CONCLUSIONS

1. The classifications of concrete crack resistance on the base of force and energetic of fracture mechanics criteria is proposed.
2. The formulas for determination of force and energetic parameters in dependence on fine concrete strength and coarse aggregate concentration are obtained on the base of experimental data.
3. Correlation between crack resistance of cement stone and crushed-stone concrete and gravel concrete is determined.

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EFFECTS OF CONCRETE CREEP ON THE ULTIMATE COMPRESSIVE STRENGTH OF CONCRETE COLUMNS

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ABSTRACT. The allowances for the long term creep effects in the British Standard and in the Eurocode 2 for the design of reinforced concrete columns have shown some discrepancies. If the British Standard for the design of a reinforced concrete column were followed, a smaller column section would be required than if the Eurocode 2 were adhered to. As a result of this, the British design approach will be more economical and thus more competitive. In order to understand fully the long term behaviour of concrete columns subjected to high stresses and to improve the level of confidence in our design, this research project has been established.

Keywords: Concrete, Creep, Strain, Ultimate Strength, time dependent effects

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INTRODUCTION

The new Eurocode 2: Design of concrete structures (EC 2) [1] has put more emphasis on the time-dependent effects on the design than that recommended by BS 8110: Use of structural concrete [2]. The design approach is similar between the BS 8110 and the new EC 2 in large except in the treatment of time-depending effects. In BS 8110, a strain of 0.0035 is treated as the ultimate strain of a concrete element, whereas in EC 2, various situations are considered and limits are set for certain situations. Sections which are subjected to compression throughout are allowed a limiting total strain of 0.002 and those that have their neutral axes on the least stressed side of the section are allowed a limiting strain of 0.0035. For sections intermediate between the two situations, interpolation between the two limits is allowed.

Most of the studies on concrete creep behaviour have concentrated on members subjected to medium or low stresses. The limited published data in studies of creep behaviours in highly stressed members have been found to be contradictory. In this study, the model columns were cast and were stressed to above 70% of their ultimate strengths. These prisms were allowed to undergo further deformation under the applied load.

Creep under very high stress accompanied by very high lateral creep will lead to internal cracking when Poisson's ratio is in excess of 0.5 [3]. This can in turn affect the ultimate compressive strength of the member.

Work by Reid et. al. [4] on eccentrically loaded columns found that creep strains about the minor principal axis due to bending are greater than those about the major principal axis and axial creep strain is less than creep strain in bending. This again supported the EC 2 approach for the design concrete member in compression.

BS 8110 ALLOWANCE FOR CREEP EFFECT

In BS 8110, the limiting strain of 0.0035 is allowed for the total strain which includes the elastic strain due to the applied load and other effects such as shrinkage and creep. A simplified approach for estimating ultimate creep is recommended in BS 8110. The approach follows an earlier method based on Comite Europeen du Beton (CEB) 1970. The ultimate creep function is expressed as:

$$\Phi_{\infty} = \frac{1}{E_c(t_0)}\left[1 + \phi_{\infty}\right]$$

where ϕ_{∞} is the ultimate creep coefficient which is obtained from Figure 7.1 of BS 8110: Part 2. Thus from knowing the ambient humidity, age at loading and effective section thickness, the ultimate coefficient can be estimated.

Where there is no moisture exchange, i.e. in sealed or mass concrete, basic creep is assumed to be equivalent to that with a volume/surface ratio greater than 200 mm at 100% relative humidity.

Although there is no provision for estimating creep coefficient as a continuous function of time, approximately 80, 50 and 30 percent of the ultimate creep will develop at a rate corresponding to an effective thickness of greater than 400 mm.

EC 2 ALLOWANCE FOR CREEP EFFECT

EC 2 treated shrinkage and creep effect as second order effects. Section 3.1 and appendix 1 of EC 2 gives guidelines on the estimation of time-dependent effects. The creep function is given by:

$$\Phi(t, t_o) = \frac{1}{E_c(t_0)} + \frac{\phi_{(t,t_o)}}{E_{c28}}$$

where $\phi_{(t,t_o)}$ is the creep coefficient defining creep between times t_0 and t.

CREEP EFFECT ON THE ULTIMATE STRENGTH OF CONCRETE

Neville [5] suggested that failure of concrete would occur at a limiting strain whether that strain is reached by rapidly applied high stress or a lower sustained stress. Neville [6] showed that for concrete loaded at 7 days and 28 days old to a stress/strength ratio of higher than 0.85, failure by creep could occur. Continho [7], found that for concrete failure induced by creep to occur in older concrete, the stress/strength ratio had to be even higher. An experiment carried out on six months old concrete demonstrated that a stress/strength ratio of 0.96 could cause the concrete to fail by creep.

Hughes et. al. [8] loaded concrete up to 25% of it's 28 day strength and maintained the load for over ten years before assessing its ultimate strength The strength of the specimens which had been subjected to long term loading were found to be similar to those which were not loaded during that period.

Roll et. at. [9] conducted tests similar to Hughes et. al.. A range of stress/strength ratios were studied and it was found that the ultimate strength of the specimens which were subjected to long term loading was increased.

Sturman [10] proposed a relationship for eccentrically and concentrically loaded concrete elements to express the fact that the maximum stress and strain could be higher in a member subjected to flexural compression than that subjected to axial compression.

EXPERIMENTS

The objectives of this investigation are:

1. To investigate the characteristics of creep in unreinforced concrete columns subjected to high stresses.
2. To study whether concrete creep could lead to concrete failures in the long-term.
3. To study whether concrete creep could cause immediate failure when subjected to high initial stress.
4. To investigate the effects of creep on the ultimate strength of concrete columns.

Unreinforced concrete prisms, 50×50×300 mm were cast for this study. After curing for 28 days in the curing tank, some of the prisms were put under an axial load in a test rig and the others were left unstressed under the same environment. The prisms that were stressed were taken to close to their ultimate strength at that stage. The load in the prisms was maintained in the rig for a period of testing.

Applied Load

The testing procedure for the long- and short-term creep effects varied slightly. For the prisms which were stressed to about 90% of their predicted strength at that age, were tested by applying a constant axial load through the hydraulic ram of an Amsler testing machine. The change of strain in the prisms was measured throughout the test. Prisms which were stressed to a lower stress/strength ratio were expected to creep without causing failure for a longer time. They were placed in a testing rig under which the compression was maintained by a spring. During the testing period, these prisms were kept in the curing room where the ambient temperature and the moisture were kept constant.

Duration Of Test

The prisms, which were subjected to spring loads, were monitored for their deformation for four months. In general, the spring loads were then removed from these prisms and they were allowed to recover for three weeks before they were load tested to failure.

Monitoring

The creep and the shrinkage in the prisms were monitored for the testing period. Prisms from the same batch which were not stressed were placed next to the axially loaded prisms to account for other effects such as shrinkage and drying creep. The difference between the stressed and the unstressed prisms gave a measurement of the creep in the prism.

Both strain gauge and Demac gauge were used to measure the creep strain and shrinkage in the prisms. Strain readings at mid-height of the prisms were presented.

Samples

Three sets of prisms were cast to investigate the effect of creep on the ultimate strength of the prisms. Four more sets of prisms were cast and were subjected to compression in a testing rig for a duration of three months. During this period of time, the specimens were kept in the curing room. In each batch of tests, three prisms were subjected to compression and another three prisms were left unloaded in the curing room to monitor any environmental effects such as shrinkage of the prisms. The prisms in the testing rig were then unloaded and were kept in the curing room for a further three weeks to allow them to recover from both elastic and creep strain.

SHORT-TERM CREEP EFFECT

Figure 1 shows the stress/strain characteristics of a prism loaded to 80% of its predicted strength and allowed to creep. The creep effect slowed down quickly after the load increment ceased. The prism did not fail in this case and the ultimate strength was as predicted.

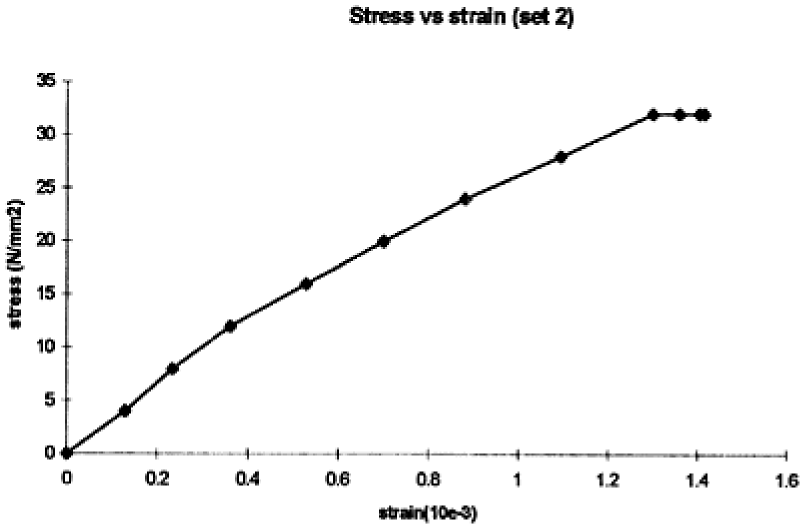

Figure 1 Stress/Strength ratio=0.8

Figure 2 shows the stress/strain plot of a prism which was initially loaded to 85% of its predicted compressive strength and allowed to creep. The compression was increased when the increment in the strain became insignificant with time; about two hours after the load was maintained. The load was increased to 90% of its predicted capacity. The load was maintained and the prism failed about twenty minutes later. The failure of the prisms was similar to that due to direct compression.

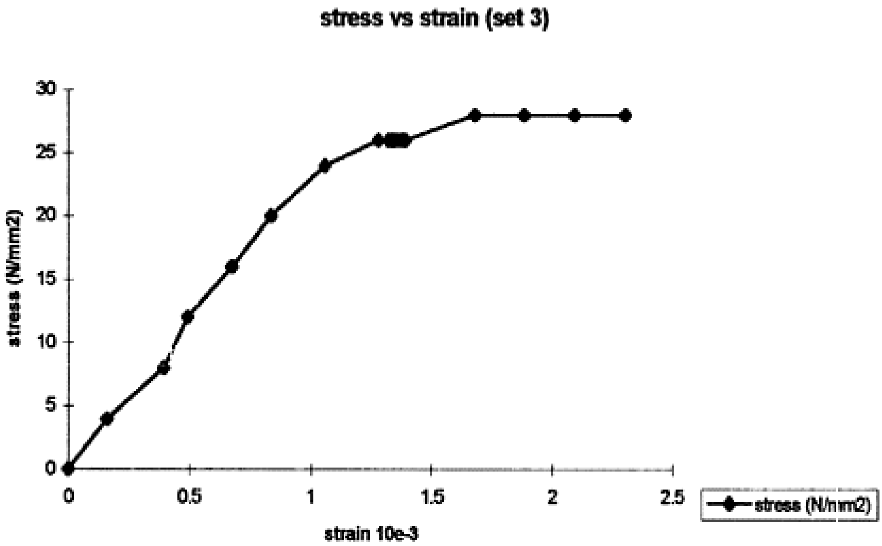

Figure 2 two-stage loading of prism

LONG-TERM CREEP EFFECT

Figure 3 shows a stress/strain plot of a set of prisms subjected to a stress of 70% of their predicted strength and allowed to creep for four months. In this situation, concrete creep has not caused failure in the prisms. The prisms recovered some of their elastic strain and creep strain when the compressive loads were removed. The load test of these prisms showed that the ultimate strength of these prisms was not different from those prisms which had not been subjected to axial compression.

ULTIMATE STRENGTH

From the investigations of the long- and short-term loading of the prisms, it appeared that creep strain could lead to failure of a compression member when its total strain exceeded a limiting strain. Where the creep strain in a member did not cause the total strain to exceed the limiting strain, its effect on the ultimate strength was insignificant.

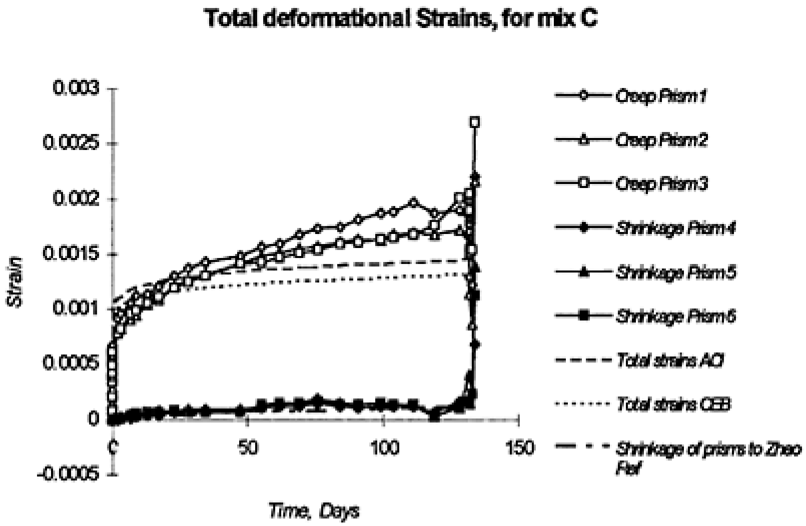

Figure 3 Prisms subjected to compression for four months and allowed to recover for one week before testing to failure.

The failure stresses of the prisms from three sets of tests are shown in Table 1. The results suggest that the ultimate strength of a prism is not significantly affected by the axial load applied previously.

Table 1 Comparison of the ultimate strength of the concrete prisms

	Failure stress of prisms which were subjected to compression before (kN/mm^2)			Failure stress of prisms which were not subjected to compression before (kN/mm^2)		
Batch 1	21.8	—	24.6	24.6	23.9	22.6
Batch 2	24.1	24.4	26.0	24.8	23.8	27.1
Batch 3	24.0	20.6	28.6	26.4	26.4	26.5

COMPARISON OF DEFORMATION PREDICTION METHODS BETWEEN BS 8110 AND EC 2

The creep coefficients obtained from BS 8110: Part 2 for the concrete prisms were found to be larger than those obtained from EC 2 [11]. That is the predicted ratios of specific creep to the specific elastic strain on the application of load were higher when applying the procedures recommended in BS 8110: Part 2.

Brooks [12] compared various methods for the prediction of concrete deformations. For mass concrete members, it was shown that the creep function predicted by BS 8110 was higher than that found from EC 2.

CONCLUSIONS

The short term loading on the prisms has given valuable information in relation to the strength of concrete columns. It has been shown that the effect of creep in a compression member could lead to failure of a compression member when the total strain in the member exceeded the ultimate strain. The allowance of an ultimate strain of 0.002 in EC 2 for members subjected to axial compression appeared to be more relevant to the real behaviour of the member.

The results from the two stage loading of the prisms have shown that the small amount of creep strain in comparison with the elastic strain is not usually the major cause of failure of the member.

The results from the prisms which were subjected to long-term loads have shown that, where the total strain in a member does not exceed the ultimate strain during its life, the ultimate compressive strength of the member is not significantly different from those which were not loaded. The members recovered from both elastic and creep strain when unloaded.

The predicted creep coefficient and the total strain obtained from BS 8110: Part 2 were found to be higher than those obtained from other prediction methods.

Further work is being undertaken to investigate the ultimate strength of concrete prisms which are not allowed to recover from elastic and creep strain.

FURTHER WORK

Further work is being undertaken to investigate the creep effect on the ultimate strength of concrete prisms which have been subjected to long-term creep effects and were load tested to failure without recovering from elastic strain, creep strain or shrinkage strain.

An assessment of the actual total strain in concrete columns designed following BS 8110 is being carried out to verify that they will not fail during their design life as a result of the total strain in the columns exceeding the limiting strain.

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HEALTH AND SAFETY IN CONSTRUCTION IN NEPAL

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ABSTRACT. Safety is the most important subject to be considered in construction industry. Unsafe working condition not only cause minor accidents or injuries, it may also cause death of workers. In the country like Nepal, the prevailing practice is not sufficient to promote and safeguard the health and safety condition of the workers. On the other hand the prevailing rules and regulations are not implemented effectively. Today’s construction, whether in developed countries and in developing countries, consume concrete. Construction has direct relationship with the development and development has been always for the service of mankind. This paper tries to focus the condition of such workers who work with concrete/cement for the service of mankind and need special consideration.

Keywords: Cement, Concrete, Construction, Hazards, Health, International Labour Standards (ILS), Labour Law, Personnel Protective Equipment (PPE), Safety.

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INTRODUCTION

Safety is the most important subject to be considered in construction industry. Unsafe working condition not only cause minor accidents or injuries but it may also cause death of workers. No compensation can provide the life for a dead person. [1] However, dangerous and risky, construction is not possible without the workers' involvement. Despite of mechanisation of construction in developed countries, manual operation are still essential.

Shortly after 1800, Portland Cement was discovered. Since then, it has become the most used cementing material in the construction industry. [2] Because of its versatile nature, concrete can be formed into different shapes as required and thus it has become an important and widely used material in modem construction.

Construction has direct relationship with development because, it fulfills people's desire for socio-economic development. Development has always been for the service of mankind. This paper tries to focus the condition of such workers who work with concrete/cement for the service of mankind and need special consideration.

GENERAL STATUS OF CONSTRUCTION WORKERS IN NEPAL

Construction workers in Nepal, basically, are peasants and are thus, seasonal. Most of them are not permanent in the trade. Annual requirement of construction workers is about 200,000. Influx of construction workers, having more skill and experience are increasing day by day from the neighbouring country, India. Construction workers both skilled and unskilled do not need formal training or license to practise. They are most unorganised and they do not have any unions. These all have made them very weak in protecting their interest and enhance the trade.

Daily wages of skilled and unskilled workers fixed by the Government are Rs. 130 and Rs. 80 (1 US$ is equal to Rs. 50.00) respectively. The prevailing wage in residential construction is Rs. 140 to Rs. 200 for skilled and Rs. 70 to Rs. 100 for unskilled. Normally, if the job is for longer duration and if the workers are working with a contractor, lower rate are paid and in the smaller projects directly handled by the owner, higher rate are paid. However, the wage obtained by the construction workers is encouraging in comparision with the other workers in other trades. Apparently the basic reasons are lack of job security, appropriate working condition and facilities, safety measures etc. Paid leave is not available to them. Except a snack break, they do not even get one day paid leave in a week. As a consequence of this, against the ILO Standard of at least 24 consecutive hours of rest per week (Convention No. 14), workers are compelled to work without taking any rest. To make more money, if job is available, they work up to 14 hours a day.

Temporary labour huts with poor ventilation and light are provided for the workers who live at construction site. Huts are built with stacking bricks or by curtaining with CGI sheets. In some huts, cement is also stored.

By virtue of socio-economic condition of developing countries, involvement of women workers in construction labour has become a common scenario and their participation in heavy manual labour has a considerable effect on their health. As revealed by the survey carried out by the International Labour Organisation (ILO) among women construction workers in Bombay- miscarriages and menstrual disorders are just two of the many health hazards these workers have to battle against. Many such females suffer from prolapse of the uterus and backaches. [3] The situation of women workers in Nepal is not so severe as the survey revealed. There still exists the general concept that heavy load carrying and overtime working are generally responsibility of male construction workers. Comparatively lighter activities like marble chips rubbing, sand screening, mortar preparation, moving concrete pan from hand to hand for concreting are the major jobs women construction workers perform. The heavy type of job they mostly perform is crushing stones manually for producing aggregates. These jobs are considered suitable for women workers.

ILO STANDARDS AND GOVERNMENT RULES

International Labour Conventions and Recommendations are adopted by the International Labour Conference, after consultation with ILO's member States. In 1966, Nepal become member of the ILO. However, none of the South Asian Association for Regional Cooperation (SAARC) countries have ratified all the 173 ILO Conventions. India has ratified 36 Conventions, Bangladesh 31, Sri Lanka 30, Pakistan 29 [4] and Nepal 5. ILO Associate Expert on International Labour Standard Tim De Meyer has commented in 1993 that Nepal has implemented only 2 Conventions. [5]

Nepal has ratified Equal Remuneration Convention 1951 (No 100) in 1976, Discrimination (Employment and Occupation) Convention 1958 (No 111) in 1974, Weekly rest (Industry) Convention 1921 (No 14) in 1986 Minimum Wage Fixing Convention 1970 (No 131) in 1974 and Tripartite Consultation (International Labour Standards) Convention 1976 (No 144) in 1994. Besides the general ILO Standards (ILS) like freedom of association, prohibition of forced labour, equality of opportunity and treatment, wages, weekly rest and paid leave and etc. there are ILS on Occupational Safety and Health which are more specifically related to the construction workers. Some of the Conventions are:

No 155: Occupational Safety and Health Convention, 1981

No 161: Occupational Health Services Convention, 1985

No 13: White Lead (painting)Convention, 1921

No 115: Radiation Protection Convention, 1974

No 139: Occupational Cancer Convention, 1974

No 170: Chemical Convention, 1990

No 119: Guarding of Machinery Convention, 1963

No 127: Maximum Weight Convention, 1967

No 148: Working Environment (Air Pollution, Noise and Vibration) Convention, 1977

The revised version of Safety Provision (Building) Convention 1937 (No 62) is the Safety and Health in Construction Convention 1988 (No 167). This Convention applies to all construction activities (building, civil engineering, erection and dismantling on a site), from the preparation of the site to the completion of the project. [6] The main aim of this Convention is to ensure safety and health in construction. As per this Convention all appropriate precautions shall be taken to ensure that all workplaces are safe and without risk of injury to health. However, such important Conventions are still to be ratified by the Government.

After the collapse of the Panchayat (partyless) system of government in May 1990, the new Constitution was promulgated in November 1990. The “Labour Act 2048” came into force on 15 May 1992 which has some special provisions related with construction industry. Before this, there was no special law regarding labour in construction industry. The law regarding “Factory and Factory Workers 2019” which was formulated under the “Factory and Factory Workers Act 2016” was the reference at that time. “Labour Act 2048” has following special provisions applicable to the construction industry:[7]

a. sufficient tools and equipment are to be provided by the management at construction site.
b. it is management’s responsibility to manage shelter, fooding, drinking water etc., if more than 50 workers are involved in a construction.
c. it is management’s duty to have worker’s insurance against accident.
d. it is management’s duty to keep construction site safe.
e. it is management’s duty to provide necessary personal protective equipment (PPE) for the workers involved in construction.

In general, “Labour Act 2048” has various provisions and are also supposed to be applicable for the construction workers also. Some of them are:

a. Working hours should not be more than eight hours a day or forty eight hours a week and there should also be a holiday in a week.
b. Government shall fix the minimum wage for the workers and no workers shall be paid less wage than the minimum.
c. Various clauses regarding health and safety.
d. Clauses regarding welfare activities such as welfare fund, compensation, rest room etc.

The provisions of the Act have an effective impact on the factory workers, but for the construction workers, they have very little practical implication. Thus, they are not found practically strong to favor the construction workers in Nepal.

CONCRETE INDUSTRY IN NEPAL

Stone, brick, timber, mud and lime mortar are the traditional building materials widely used in Nepal. Concrete construction in Nepal started about half a century ago. Palace of then Prime Minister Mohan Sumsher (April 1948-November 1951) is considered as a first concrete structure in Nepal. The technology used in this structure was influenced by the British Indian technology prevalent in the then India. At the beginning concrete constructions were a status symbol. Government buildings were built in accordance with it. The early application of the concrete were limited to the slab-like structures.

Gradually, people became aware of the inherent properties of the concrete-strength, durability and easy to cast into desired shape. Tradesmen got opportunity to be trained in concrete structures. This attracted ordinary people towards concrete structures.

Deforestation limited the availability of timber. Cost of the materials were also increased. On the other hand, Nepal being a mountainous country, aggregates are available in plenty. Furthermore, better performance of the concrete stimulates people towards the use of concrete in construction.

Concrete industry in Nepal is yet to be mechanised. Mixing, transportation, pouring etc. are performed manually. However, concrete mixer, hoist etc. can be seen in some constructions.

HEALTH AND SAFETY IN CONSTRUCTION

Most of the large/major constructions in Nepal follow the FIDIC Conditions of Contract or WB guideline for procurement. There is not uniformity in Conditions of Contract applicable to constructions. Sometimes, only simple agreement works between the Owner and the Contractor. It is also found through observation that even the Department of Building has not prepared detailed conditions of contract and specifications. In such a situation, effective implementation of health and safety standards are hardly possible. No construction site is inspected by the officials from the labour office and in general, no Safety Officer is appointed in the construction site.

The form work and false work are not properly designed. In March 1994, form work and false work of a boiler house of a five star hotel in the capital city collapsed during concreting and claimed the life of a person and eleven others injured. [8] Similarly in September 1994, while removing formwork from the newly casted roof of a school building, whole of the slab collapsed. [9] There are many such unreported cases.

However, before concreting, the formworks are inspected by the engineer. There is also no practice of hammering the nails exposed on the planks immediately. Piercing by rusted nails are common and such happenings are not reported in any form.

Concrete mixer is used in mixing concrete. Pumping system is yet to be introduced. Thus, concrete is kept in an iron pan and moved hand to hand to the place of casting. Personal protective equipment such as boots, gloves, protective clothes are not compulsory to all the workers. Some workers are seen with rubber boots but not all. Some roll polythene sheet on their foot during concreting. Patches of cement slurry are seen on the body of the workers.

The workers are not only untrained they are not orientated with the common hazards and probable safety measures to be adopted. They are unaware of this. Working with concrete does not exclude them handling cement. Government owned cement distributing company operates from a godown with very poor ventilation and lighting. The porters involved in loading and unloading of cement are found fully covered with cement dust. They are not provided protective equipment like clothes, boots, gloves and masks. The floor is found covered with a thick layer of cement dust.

CONCLUSION AND RECOMMENDATIONS

Construction is a complex process and is considered to be dangerous and risky job. However, workers' involvement in the construction job is essential. In modern construction, concrete is an important and useful material. It has served mankind in a significant way. Most of the large/major constructions in Nepal adopts documents of international standard such as FIDIC or WB. But the implementation part is not encouraging. There is no licensing system for workers. Thus, there are untrained and unaware workers in construction. Since they are most unorganised, they are not able to form a union to protect their interest.

In most of the ICB contracts, simple PPE are seen provided to the workers. Insurance is also done. However, there are significant number of large residential, commercial and other construction which do not follow the safety measures. Dust is also a common nuisance on any construction site. It may arise due to the handling of earth, aggregate, cement, traffic and wind. The dust inhaled by workers at construction sites may cause lungs diseases like Silicosis, Talcosis, Graphite Pneumoconiosis, CNSLD etc. [10]

Occupational diseases are the results of physical conditions and the presence of industrial poisonous and non poisonous dust in the atmosphere. Raw materials, products, by products and waste products may, in the process of being extracted or manufactured enter the body in such quantities as to endanger the health of the workers. [11] Workers in potteries ceramics, metal grinding, refractories, slate, pencil mines or stone mines, road building, construction works are affected by respiratory diseases, leading to spitting of blood and a painful death due to Silicosis. [12] Workmen handling bulk cement, lime or fine pozzolona shall wear protective clothing, respirators and goggle shall be instructed in the need of cleanliness to prevent dermatitis and shall be provided with hand cream, protective jelly or similar preparation for protection of exposed skin. [13]

For the development of the country and community and to provide required services for the mankind, construction is an essential and basic activity. Modern construction whether in developed and in developing country consume concrete. Thus, the workers involved in construction and related works particularly in the countries like Nepal where no safety practices have become a system need special consideration. Some of the recommendations are as follows :

1. Training shall be made compulsory.
2. Licensing system shall be introduced. This helps maintaining the register of construction workers.
3. Inspection of construction sites by the 'Health & Safety Inspection' (HSI) from the labour office is essential. For such inspection check list should be prepared In the involvement of ILO representative, workers representative, government's representative, construction engineer/manager, contractor etc. checklists for different activities shall be finalised so that the prepared checklists can be practically applicable.
4. Types of formwork should be specified and forms should be as per drawings provided and should be errected accordingly.
5. Form work should be designed to carry vertical and horizontal loads including the dead weight of shuttering and scaffolding, concrete mix, reinforcement, persons performing concreting, equipment for transporting concreting horizontally, vibration and others.

6. Personal protective equipment should be made compulsory.
7. Every five year construction workers shall have access to the medical examination for timely detection and/or for timely treatment of the hazards.
8. In each large construction (employing more than 100 workers per day) should publish a "construction bulletin" at least once a month in a local language to make the construction workers aware of the nature and the risky/hazardous part of the construction where they are needed to be cautious and the PPE should use.

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FOUNDATIONS FOR LOW RISE BUILDINGS IN ROMANIA AND UK: COMPARISONS AND EFFECTS

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ABSTRACT. With the changing political and commercial philosophies now occurring in parts of central and eastern Europe, the potential for mutually beneficial collaborative research projects between nations has recently substantially increased. In the construction technology field, there are enormous opportunities to undertake applied comparative research to compare codes, standards and changing construction trends. One such international research project is currently exploring the construction of concrete foundations for low rise residential properties built in the UK and in Romania. The focus of the research is to ascertain the incidence and causes of foundation failure when broadly similar properties are constructed on apparently identical ground type, but using different foundation systems.

The paper will report on aspects of this research, with reference to the technical, quality and other administrative procedures to show the differences in approach in the selected countries.

Keywords: Residential development, foundation failure, legal requirements and quality procedures in UK and Romania.

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INTRODUCTION

Changing political climates in countries can impact on society and across the world in many different ways. The effects of change can take time to percolate through a nation or in some instances such change can be swift. The transition from one political system to another brings demands that are sometimes painful, especially when a political system has been in place for decades and urgent changes are demanded. Romania is one such country. For over forty years, Romania functioned under a command economy political system, then in 1989, a revolution occurred. Since 1990, Romania has sought to adopt a western style of market economy. The process of change has not been easy for the country as a whole or for its construction industry. Commercial pressures, of the kind previously unknown, have brought with it many different stresses.

Despite frequently held opinion to the contrary, Romania is a technologically sophisticated country. Romania's construction industry, through political dictat, was forced to provide mass housing for a rapidly growing population and the migration of people into cities and towns, because of the socialist industrialisation. Low and medium rise blocks of flats were, of necessity, constructed to satisfy this demand. As there were suitable, locally available raw materials and an abundance of labour, it was inevitable that concrete would become one of Romania's primary building materials.

The form of residential development in Romania in the last five years has changed. The construction of blocks of flats has given way to two and three-storey housing units. However, the expertise of Romanian engineers is still reflected in the design and construction of these units and the whole is the subject of an international research project based in the UK.

This paper will look at the nature of housebuilding in Romania and make some comparisons with the housebuilding process in the UK. It will identity aspects of building control and associated legislative issues when considering the construction of foundations in concrete.

THE NATURE OF RESIDENTIAL DEVELOPMENT

There are some striking similarities between aspects of housing construction in the UK and Romania. In the UK, there was a phase during the 1960s—1970s where the demand for social housing was high and the authorities deemed that a speedy and efficient method of providing these was by the construction of blocks of flats. This strategy was the same in Romania, but the timeframe was extended to include most of the 1980s too. A difference in approach is conspicuous in that in Romania during this period, the vast majority of dwellings were constructed of blocks of flats, whereas in the UK, there was a private housing sector developing to satisfy the demands of individuals wishing to buy their own property. In Romania before 1945 and the onset of a communist regime, there was a housebuilding industry constructing dwellings typically of single and two-storey traditional brick construction. Further, since the revolution in 1989, Romania has ceased to build new blocks of flats in preference to contemporary, single and two-storey housing in a detached, semi-detached and terrace form. In this way, therefore, a comparison between UK and Romanian residential development is both timely and relevant.

A major international research project is underway which is seeking to compare and contrast aspects of residential development with a particular focus on their foundations and the failure thereof. The comparison between foundations used on properties up to three storeys in height only, provides an ideal focus to identify similarities and differences both in design and construction method.

FOUNDATIONS IN CLAY SOILS

In the UK, the subject of foundations for low-rise residential development in shrinkable clay soils has been the subject of much research and debate for many years. The debate has often centred on the depth at which foundations should be placed in shrinkable clay soils. Foundation depth is important because of the effects that seasonal changes can make to expansive clays. This is exacerbated in the UK when there are periods of severe drought as have occurred in 1976, 1984, 1989, 1991 and 1995. Trees too can significantly affect shrinkable clays. In the UK, the failure to construct house foundations at a suitable depth has meant that there have been numerous foundations which do not support the superstructure properly because they have been disrupted, cracked, or been subjected to undue stress for which they were not designed. In essence, therefore, these foundations have failed and underpinning is necessary to rectify this damage. A simple statistic is that in 1991, the value of insurance claims for such remedial works was £540 million. It is unnecessary to expound upon the typical details of UK foundations in shrinkable clay soils, apart from plain (unreinforced) concrete is often used.

In contrast to the way UK foundations are constructed, the construction adopted in Romania for two-storey houses on shrinkable clay is rather different. Firstly, in all cases, foundations for every building are purpose designed. The minimum foundation depths used varies depending upon the water level. The minimum depth of foundation is 1.5 metres, although this is in some instances increased to 2.0 metres. In addition, all the foundations are constructed of reinforced concrete. Whilst such specifications are not particularly complex, it is important that this difference in construction is noted. The subject of legal and quality controls, too, is important.

LEGAL REQUIREMENTS AND QUALITY PROCEDURES

In both the UK and Romania, legal requirements and quality procedures exist to ensure concrete foundations meet the agreed standards.

In the UK, housebuilders normally construct houses which, upon satisfactory completion, a ten year warranty is provided to housebuyers against major structural defects. To satisfy the requirement for the granting of a warranty, housebuilders agree to build in accordance with the Building Regulation [1] and, for example, the Standards [2] of the National Housebuilding Council (NHBC). Together, these requirements specify steps to be taken for constructing foundations in clay soils, or other difficult ground In some instances, it will be necessary to undertake extensive ground investigations and submit for checking proposals for foundations in such ground. During the construction stage, inspections will be undertaken by field inspectors employed by the NHBC and/or by the local authority. Once the work is approved, the housebuilder may proceed with subsequent construction work. If the work is not satisfactory, the housebuilder will be required to rectify the work before proceeding. Although materials could be tested by the inspectors, this is seldom done. If the ground conditions are different at the construction stage than anticipated at the design stage, it will be necessary for a new foundation design to be produced. In practice, however, many of the strip foundations for two-storey houses are of plain unreinforced concrete. The depth at which such foundations are placed, usually 0.8–1.0 metres, is determined on-site following a field inspection of the ground. Where clay soils exist, the depth will generally be not less than 1.0 metre. This is in marked contrast to the Romanian requirements. The width of foundations are determined by normal design code requirements, but a compressible material or void former would be placed on the vertical side of a foundation as a precaution against heave.

The UK regulations lay emphasis upon the appointment of an engineer to advise on technical matters where complex ground conditions are anticipated. Some guides and codes are written in non-technical terms to assist the housebuilder to identify poor ground, whereupon he will be responsible for taking the appropriate action. If a housebuilder fails to comply with specific requirements, the ten year warranty could be withheld. In the case of local authority control, the officers have the power to serve a notice to expose any building work to ensure compliance. However, where an authority serves such a notice, the housebuilder is entitled to appoint an independent expert. When an action is taken through the courts, a housebuilder can be fined for the error and have imposed an additional fine based at a daily rate on the number of days the defect remains. In practice, whilst this power through the courts exists for local authorities, it is seldom pursued. There are two reasons for this. Firstly, each local authority has its own policy and most prefer to determine solutions to such problems by negotiation rather than by legal means. Secondly, and significantly, if a local authority undertake to expose building works in the belief that defects exist and subsequently find that the work is not defective, then it is responsible for all reinstatement costs.

In Romania, the procedure for checking the concrete in foundations are clearly stated in specific codes and requirements on the use of materials and components, and in the Romania Law of Quality [3]. It sets out specific duties, responsibilities and liabilities of individuals and agencies associated with the construction of any building. These responsibilities fall upon investors, designer/engineer, contractor, building owner, building administrators and building users.

When considering a building, an independent chartered design engineer must certify that the proposal has been designed properly. During construction, the concrete must be tested at different stages. The contractor must undertake concrete tests. The supplier, too, must certify concrete strength in accordance with the design. Site tests are required upon receipt of the concrete and at the point of pouring the element (eg the foundation). Independent and separate external quality checks may also be undertaken by appointed inspectors under contract to the government. These inspectors may carry out any test on the concrete and they also have the authority to check the validity of all the tests executed by others.

The penalties of non-compliance with the law are specified in the law and these are dependent upon the contraventions and the consequences of the non-compliance. The penalties range from small fines for minor contravention to large fines and a term of imprisonment of up to twenty years for major contraventions. An interesting aspect of the government paid inspectors is the concept of commission payments. This is based upon the ability of an inspector to bring a successful challenge against a builder. Where an individual or agency is found to contravene the law of quality, the inspector will be paid a small percentage of the imposed fine. The inspector therefore imposes a specific fine in accordance with a prescribed defect or contravention. This fee is in addition to the payment received for executing the role of inspector. The idea behind the commission—based fee system is to ‘reward’ the inspector for ensuring high quality is achieved. Where the contravention is serious, a Commission is appointed, consisting of specialist individuals whose role is to seek to establish blame. It is this Commission only that would impose a custodial sentence on a builder.

DEFECT ERROR SOURCE COMPARISONS

A comparison between the UK and Romanian causes of error based upon currently available data is difficult. Information gained from two sources [4,5], one Romanian and one from the UK, provides an unrefined comparison between similar sources of error as shown in Table 1. The Romanian data is based on a survey of 840 cases of building defects, whereas the UK study is based on a survey of 360 cases of residential properties known to have inherent structural defects.

Table 1. Comparison of Defect Error Sources.

Defect Source	**Romanian %**	**UK %**
Design mistakes	16.6	59.2
Construction mistakes and material defects	56.7	6.8
Error in building use	14.4	15.2
External factors	12.3	18.9

An analysis of the data reveals that design mistakes are considerably more prevalent in the UK study than in the Romanian study. This result demonstrates the apparent detailed attention paid to design by Romanian engineers compared with UK designers. Conversely, the data suggests that the execution of the works is more problematic in Romania than in the UK. One of the major problems in Romania is the lack of high quality materials, especially for finishes and insulation. The net effect of this is that, whilst there is adequate and effective supervision on site, the failure often stems from a premature deterioration of low quality materials and components. Another possible reason is that it is only on major building projects in Romania that modern sophisticated plant and equipment is available. In the UK, high quality materials and components are readily available. These coupled with competent supervision and the wide use of plant and equipment, is likely to be a reason for lower construction mistakes and material failure. Errors in use and external factors are similar in each of the survey results. It would be unwise at this stage to draw any detailed conclusions from the Nottingham research project. However, it is worthy of note that the design/construction aspects of projects produce different outcomes with respect to damage/defects sources.

CONCLUSIONS

The type of housebuilding in Romania and the UK is outlined, together with the problems of building on clay soils. The design requirements and the control procedures in Romania are arguably more demanding than in the UK and the design errors from the table support this view. The construction aspects are difficult at this stage to quantify, but there appears to be a problem gaining access to, and the use of, high quality building materials and components in Romania.

In both the UK and Romania, legal and quality procedures exist. The UK model has been in place for many years and has evolved over time. The Romanian model and the recently introduced law cannot yet be fully evaluated because insufficient time has elapsed to judge how and if the control procedures are effective. It can, nevertheless, be stated that the sanctions available to aggrieved clients and the government are more onerous in Romania than in the UK.

This paper has not sought to provide a detail of all controls and differences in each of the countries, but merely to highlight that differences do exist. It is easy for one particular country to believe that it has the ‘better’ construction industry compared to another. What is important is for building professionals to realise that each can benefit from knowing and understanding how others operate. Only then will this more catholic approach to knowledge acquisition be beneficial and improvement realised.

Concrete is a universal construction material and Romania and the UK have a wealth of experience in its design and its use. The sharing of ideas, design techniques and construction strategies will help to ensure that defects to foundations will be minimised. It is clear from the evidence that no one country has total knowledge of the subject, but it might be possible if construction professionals realised that through dialogue and interaction, improvements can be achieved which can only benefit society at large.

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SITE WORKERS—THE CHALLENGE

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Appropriate Concrete Technology. Edited by R K Dhir and M J McCarthy. Published in 1996 by E & FN Spon, 2–6 Boundary Row, London SE1 8HN, UK. ISBN 0 419 21470 4.

ABSTRACT. The paper reviews the aspects of suitable training for site concrete workers. Based on a combined research and construction experience, a series of example training aids using models and special graphics are presented. The current development of an integrated secondary to tertiary education for building trades in New Zealand is outlined.

Keywords: Training, Site Workers, Qualifications.

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INTRODUCTION

The focus of research and development for concrete continues in the direction of analytical research and design data with an ever increasing accuracy of test performance to demonstrate the improved concrete properties. While this research growth to evaluate and improve the performance of concrete structures is a pre-requisite for development in the 21st Century, how much effort is focused upon the education of the site worker?

Most of these recent developments result in “new” concretes for the specifier. Does the site worker recognise this “new” concrete? Does anyone on site know what concrete with a Dc of 10^{-13} means? Does anyone actually care? Does anyone know that workmanship matters can change the Dc of 10^{-13} to Dc^{-10}? Many developments will never achieve their researched potential without some radical upgrading of construction practice education. Over the past 20 years the NZ Concrete Research Association, now Cement & Concrete Association of NZ, has had a hands on approach in relation to the research it carried out because it also had direct access to site workers through the provision of its construction education programme.

The key issues are practical training techniques that capture the attention of the site worker, so that although they may never understand the units of Dc 10^{-13}, they do know the consequences of their actions in terms that can be understood.

During the last 70's and early 80's, matters relating to durability were under serious discussion. The NZCRA, like many other research agencies, carried out a series of durability studies. These studies were invariably limited in terms of any statistical approach, but were designed to test overseas findings. During this period, water permeability followed by chloride diffusion permeability were test methods used to evaluate the concrete properties for durability.

In the testing regimes adopted, a component was cast in concrete using standard formwork and compaction equipment on specimens obtained by coring. This sort of approach produced a greater variability of the results compared to concrete cylinder sampling. Different curing regimes clearly showed through on the results. While these results are no different to many other researchers, it became clear that workmanship on the construction site was a considerable parameter influencing the real performance of concrete in the field.

The 1990 attendance at the Dundee Conference was made with a view to considering the international preferences for the measurement of permeability. In reality the worry of the size of the workmanship influence became more important than the concerns of methods of property measurement. Accordingly, the challenge became one of trying to influence site workmanship to meet both past and present and new requirements for concrete construction. The challenge was made significantly more difficult certainly in New Zealand, by the significant loss of independent site supervision, responsibility losses by sub contracting and ironically by the efficiency of the local ready mixed concrete industry. In this latter case the skills of understanding concrete became more and more focused away from the construction site, as the skills for concrete production were removed from the site worker. In recent days the ready mixed concrete industry has realised the difficulties that can arise from having its product abused by construction placement and is actively co-operating in the new training programmes.

OVERVIEW OF SITE WORKER TRAINING REQUIREMENTS

In the early 70's a Concrete Construction course was offered by Technical Institutes of the day, but after several consecutive years of no students, the NZ Vocational Training Board abolished the course leaving only isolated topics within a number of courses, e.g. Civil Engineering, Carpentry, Bricklaying etc. None of these courses were suitable for the site worker. In an effort to redress the balance the correspondence course developed by the then C&CA (UK) for the London City & Guilds Concrete Technology and Construction was introduced into New Zealand run by the Concrete Research Association. Even at this time the course continues to be run. The mainstay of technical adult education, as opposed to tertiary education, was undertaken through 1 or 2 day training courses. 500–600 people a year were trained in this manner in the early 80's.

Who were the people being trained? A significant proportion were clerks of works and works foremen for the principal companies. As has already been mentioned the clerk of works influence has been significantly eroded, meaning that one avenue of the transfer of knowledge to the site worker has gone. Many companies choose to subcontract the concrete placement and that transfer of responsibility causes another avenue of instruction to be lost.

The New Zealand Government has embarked upon an ambitious integrated programme which allows industries to develop training modules to suit its particular needs. The training may embrace school, polytechnic and University activity. The development of such a co-ordinated programme is proving to be a difficult task, not made any easier by the use of educationalists to write the training achievement requirements. However, out of this will come an opportunity to provide site workers with some achievement stepping stones. What has to be guarded against is setting unrealistic academic achievements in relation to practical skills achievement, i.e. we do not require a treatise on how to use an internal poker vibrator, we want to have demonstrated the practical use of the equipment. An overall industry strategy was considered in advance of the education/vocational training schemes being pursued by Government agencies. It is illustrated in Table 1.

Table 1 Draft Strategy for Training

	Advanced			Research
		National Certificate Level		
3	High	High	Special	Special
2	Mid	Mid	Mid	Mid
1	Basic Concrete Technology	Basic Construction Technology	Basic Concrete Production	Basic Testing
	A	B	C	D

It is set up in such a way that after some common core topics, there are a number of paths to suit the avenues of specialisation with the industry to achieve a National Certificate.

However the real concern is not to over emphasise these people's training at the expense of the site worker. Hence the teaching/skill achievements etc. in Level 1 are geared entirely for the concrete worker. The training given at this Level 1 is to be linked into the "system" which will allow personal development through the other training levels, but there is no compulsion element to proceed further. The skills achieved in Level 1 topics are formally credited.

Training Programme—Concrete Worker

It is the development of the Level 1 training that principally is the concern of this paper, because it directly tackles the training of workers who are in the business of placing concrete. An outline of the principal topics covered by on-site training is listed in Table 2.

Table 2 On-site Training Sessions

Level 1
Concrete Worker
Basic Concrete Technology
Concrete cover to Reinforcement
Handling and Placing
Compacting
Finishes
Curing
Weather Conditions
RMC Supply
Site Sampling and Testing

Table 3 Additional Specialised Topics

Level 1
Special Topics
RMC Drivers
Batching
Steel Fixing
Formwork
Propping
Duct Grouting
Plaster Preparation
Floor Screeds
Masonry Grouting
Block Paving

Each topic is packaged in a presentation of 1 hour which will be discussed later. Special extension topics can be added to suit particular industry needs. Table 3 shows some of the extension topics which are also treated as a 1 hour presentation. Many of these topics will have Level 2 and Level 3 components.

Training Techniques

Up until the late 80's all our training was addressed to people who were substantially literate. At that time a major New Zealand contractor who had had many of its foremen trained by the 1 and 2 day courses, asked for a training programme for its site workers—not just the foremen.

This brought about a significant change in approach. Previously it was reasonably easy to justify why something should be done in a technical sense and follow this with an overview of how to do the work. The challenge was now to explain the WHY in terms acceptable to the worker as well as HOW in real terms. This has to be done, not in the classroom environment, but in the site hut or on-site. It has to be done not totally relying on the written word, but principally based on a visual and participating approach. The training video, while useful to reinforce a point, remains impersonal and often too small scale for visual impact.

Accordingly each topic was treated in a manner that had a mixture of hands-on, model demonstration and overhead projection animation to fix the WHY in peoples minds. When some one knows WHY they are doing their job, it is much more likely they do it without the need for constant supervision of the HOW.

It is obviously not possible within the confines of this paper to cover all the topics, but a selection of model and overhead projection animation examples are given to illustrate the techniques used to get the messages accepted within a short time span of 1 hour for each topic.

Example 1 Model/Overhead Projector (Figure 1)

Concrete Compaction—Size and spacing of internal vibrator

Method (a)	Using masking tape, set out the plan view of the component on the floor of the hut or table.
	Using cardboard discs (dinner plates for smaller vibrators) get the workers to space out the discs to cover the plan area.
	Another practical feature that may need to be included is the reinforcement.
Method (b)	Using an overhead transparency project the component shape, by adjusting the projector position in relation to the screen it is possible to display the actual size of the component the worker will see.
	Using translucent scale discs cover the plan area.
	Again reinforcement may need to be shown.
Result:	Choosing the correct diameter vibrator for appropriate parts of the job and the correct spacing of insertion.
	How long? Count to fifteen.
	When do you know its okay—bucket of fresh concrete and put the vibrator in it—okay when air bubbles cease to appear.

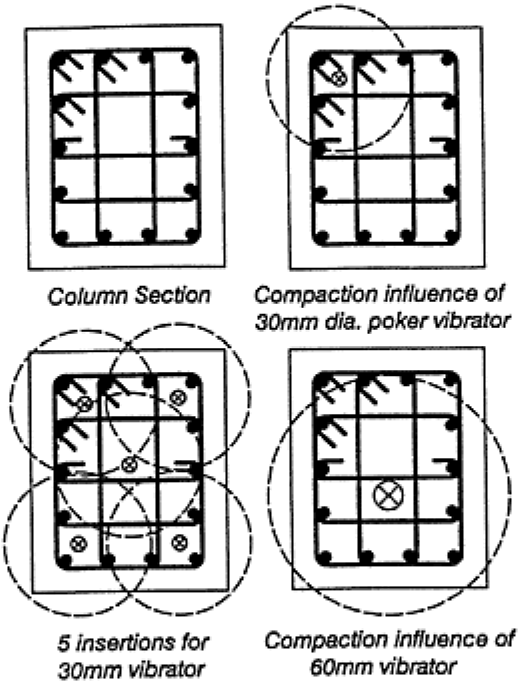


Figure 1 Concrete compaction—Size and spacing for internal vibrator

Example 2 Model (Figure 2)

Suspended Concrete Forms. Importance of central loading transfer for props and forkheads.

Materials: 1 length of dowel/1 sponge

Put runner on sponge/dowel in centre of runner, load with standard weight. Second stage put dowel outside centre thread of the runner, load with standard weight—allow model to collapse.

Repeat with dowel fitted with baseplate.

Use dowel with forkhead fitted—load centrally.

Place load on extremity of forkhead, allow model to flex and collapse.

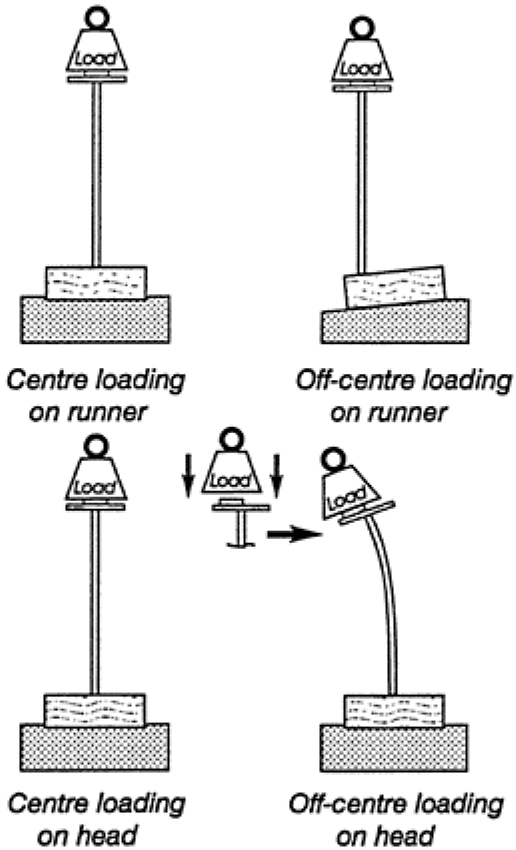


Figure 2 Importance of loading positions on props or scaffolding

Example 3 Overhead Projector (Figure 3)

Angle of keyway in floor joints

Make cardboard cutout of joints, one with heavily sloped demoulding insert, the-other with limited slope.

Put on projector to throw up shadow. Pull joint apart for shrinkage movement, then bring them into contact.

A series of joint details dowel, aggregate interlock etc. can also be clearly demonstrated by this technique.

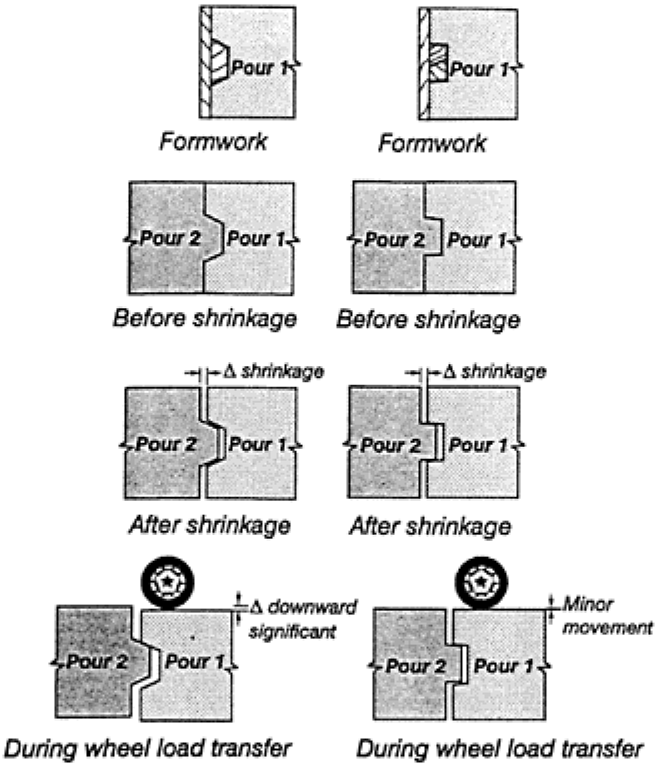


Figure 3 Keyway shape influences load transfer

In the presentation it is particularly important to emphasise that each action has a real impact in the final performance or life of the concrete member.

Some of the results of the approach can be gauged from contractor observations as to what took place after their workers attended the on-site one hour training on the different topics.

1. High quality finished required on concrete columns: Concrete gang refused to put the concrete into the formwork because they would see the formwork was going to leak.

   The formwork was rebuilt and a job requiring no remedial touch ups was achieved.
2. Measureable improvement in productivity and quality with a reduction in overseeing management, i.e. because each worker knew WHY the job he was doing was needed, there was less need for "nagging" from others to do the job properly. There was much less "remedial work".
3. Even curing got done! Very important when not doing it can change a $Dc10^{-13}$ to $Dc10^{-10}$.

By providing information in a manner acceptable to the workers, self motivation became a reality. In this regard the fact that the worker could, in many cases, see the results of their endeavours provided a sense of achievement. It highlighted the fact that you do not need to be a qualified brain surgeon to understand the impact of achieving something with your own skill level.

The “Man on the Job” series produced originally by the C&CA UK, now reproduced in a new series by the BCA (Ref. 1) represented a new reference point. A Site Concrete booklet series was produced by C&CANZ (Ref. 2) drawing upon the work of the BCA. However, it was considered to be too “wordy” for the site worker and a series of posters were developed to illustrate visually the main points of the booklets. See Figure 4.

SCHOOL/INDUSTRY TRANSITION

The danger of “condemning” people early in their lives is very real. Often entry to any form of tertiary education is bedeviled by a long list of academic requirements. This can often debar people with essentially practical skills from even entering or taking advantage of tertiary education. The result is that no further training takes place.

Polytechnic courses are often more related to academic skills before getting to grips with the practical hands on skills.

A pilot scheme is being developed by one New Zealand Polytechnic which permits entry from school into a pre-employment Building Trades Course. Students can join the course at different times during the year, which allows for easier transition from secondary education to tertiary. School leaving ages are generally being raised, but unless “meaningful” education is provided, often the extra year becomes a mindless waste of time, particularly for those people whose future paths lie in practical skills.

Entry to traditional apprentice trades is also becoming shackled with academic success requirements. The pre-employment general building skills course also provides an alternative bridge to reached practical skills equivalent to the former apprenticeship approach. A student can, after attendance at the pre-employment course which covers a wide range of topics, choose entry into the more specialised areas of construction.

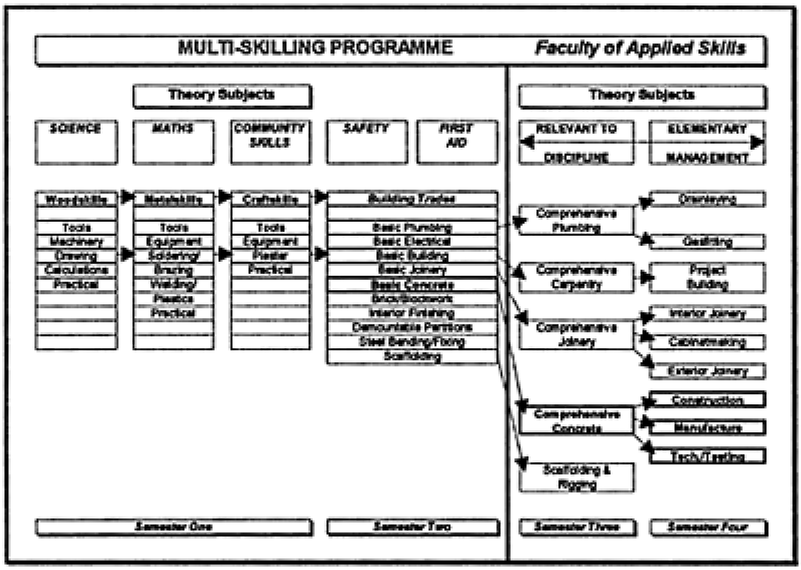
However the real challenge has still to come, because unlike specific apprenticeships of the past, these students will be Multi Skilled to a significant level in many trades.

The outline of the developing programme is shown in Table 4.

CONCLUSIONS

1. It is important to recognise that workmanship skills are a vital part of concrete performance chain.
2. Simple training properly addressed to the workers involved can significantly influence workmanship standards.
3. With research introducing new applications of materials, there is as much need for providing appropriate instruction at worker level as there is at specifier, designer or concrete producer levels.

Table 4 Multi-Skilling Programme



It is difficult within the context of an international conference to express in words a technique of training where "words" are not the primary medium of influence. However, it is hoped that the need for appropriate worker training as it influences the performance of concrete will be noted.

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Figure 4 Using a Poker Vibrator

COMPUTER-BASED CONCRETE SPECIFICATION

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Appropriate Concrete Technology. Edited by R K Dhir and M J McCarthy. Published in 1996 by E & FN Spon, 2–6 Boundary Row, London SE1 8HN, UK. ISBN 0 419 21470 4.

ABSTRACT. Concrete specification is a task common to all construction projects. Unlike other construction materials, the behaviour of concrete at the time of construction is very different from that in its later life. Concrete specifiers are predominantly concerned with a concrete's hardened performance whilst the purchaser (contractor) looks for particular fresh properties. Combining both of these requirements into an unambiguous specification for processing by the concrete producer is a complex operation, which when done well, provides construction economies, good appearance and the desired structural performance and durability. This project will deliver a computer program designed to help specifiers and contractors to produce appropriate specifications—for both fresh and hardened properties—in accordance with current construction Standards and Regulations. The project has the support of the Department of the Environment under their Partners in Technology programme.

Keywords: Expert systems, Harmonisation, Ready-mixed concrete, Regulations, Standards, Specifications.

Eur Ing Anthony Osborne is Project Manager for the Ready-mixed Concrete Bureau, *RCB*, a joint venture by British Aggregate Construction Materials Industries, *BACMI*, British Ready Mixed Concrete Association, *BRMCA,* and the British Cement Association, *BCA*. He is a Chartered Civil Engineer and has been with the RCB since its formation in October 1991. Prior to this role, his background was in civil engineering construction and design, where he gained broad general experience but with an emphasis on piling and marine works.

Andrew D Pullen is a Research Fellow at Imperial College of Science, Technology and Medicine and is Technical Manager for the Concrete Research and Innovation Centre, *CRIC*. His research interests include the tri-axial behaviour of concrete under dynamic loads and the application of IT in concrete technology.

Dr John B Newman is Head of the Concrete Structures Section and Director of CRIC at Imperial College, London. He has supervised many programmes of research including investigations of concrete durability, structural behaviour and the development of special concretes and components. As a consultant and expert witness he is primarily concerned with the development, acceptance and testing of construction materials and components and investigations into the integrity, repair and maintenance of buildings and structures.

INTRODUCTION

The correct specification of concrete is a time-consuming and often repetitive task for all, even the specialist. Recent amendments to cement standards to harmonise with Euronorms have complicated the topic further for practising engineers. Later this decade, BS EN 206 : *Concrete—performance, production and compliance criteria* will be released as a national standard (anticipated to be released in early 1998) and many concrete specifiers will find themselves tasked with responsibility for completely re-drafting their specifications without possessing sufficient knowledge of the new standards for concreting materials and accepted best practice. The objective of this project is to develop an effective method of concrete specification covering current and future standards and regulations using state-of-the-art computing techniques.

In operation, the specification system will be designed to be easily used by practising concrete specifiers (principally consulting engineers) and to be readily up-datable to provide for the introduction of future amendments to both UK and European standards and Regulations. The result of the project is intended to alleviate the burden on concrete specifiers whilst enabling the efficient production of correct, concise and appropriate concrete specifications. Benefits in quality assurance will be attained by ensuring that the current version of the appropriate Standard and Regulation will be used and that the criteria for decision making will be logged to provide an audit trail.

The project is a collaborative activity that has attracted approximately 50% of its funding from the Department of the Environment under their *Partners in Technolog* y programme. The residual funding is provided by the Ready-mixed Concrete Bureau, the project proposer and lead partner. Other partners supporting the project are Posford Duvivier, Consulting Engineers, *PD*, together with the Concrete Research and Innovation Centre at Imperial College of Science, Technology and Medicine, *CRIC*.

Development work commenced in mid-1995 with a planned completion date of August 1997 to coincide with the projected agreement on a prEN 206 for CEN enquiry. Consequently, at the time of writing much of the detailed aspects of the program's operation are unavailable. However, a demonstration system in accordance with BS 5328 is intended to be available for the Congress.

PROJECT OVERVIEW

Scope

The scope of this project is to produce a computer program to aid the production of economic, concise and unambiguous concrete specifications in accordance with the appropriate prevailing standards and regulations. Of particular importance to this project is the inclusion of BS EN 206 : *Concrete—performance, production and compliance criteria* in the program to promote its introduction to UK construction practice.

In addition to providing a means of specifying concrete, it is intended that the system will also aid the dissemination of background information on key aspects of concrete technology and practice; eg cement selection and behaviour, concrete mix design, admixtures, workability/cohesion, placing methods and site quality control etc. This part of the program will be designed to be accessible from two routes;

- as part of an on-line help facility
- as an interactive reference guide.

To maximise user acceptability and accessibility, the program will be developed for deployment on a range of popular computer operating systems. For ease of use the concrete specifier will communicate with the program through a graphical user interface, *GUI*. The adoption of a GUI provides for such items as menu bars, pull down lists, check boxes, text entry boxes as well as facilities for user feedback/confirmation with illustrations and photographs.

Methodology

System design involves development in three distinct areas;

- the knowledge base—using information from British and European Standards and Regulations (eg. BS 5328 [1], EN 197 [2], prEN 206 [3], Specification for Highway Works [4], etc.) communicating with the GUI as shown in **Figure 1**.

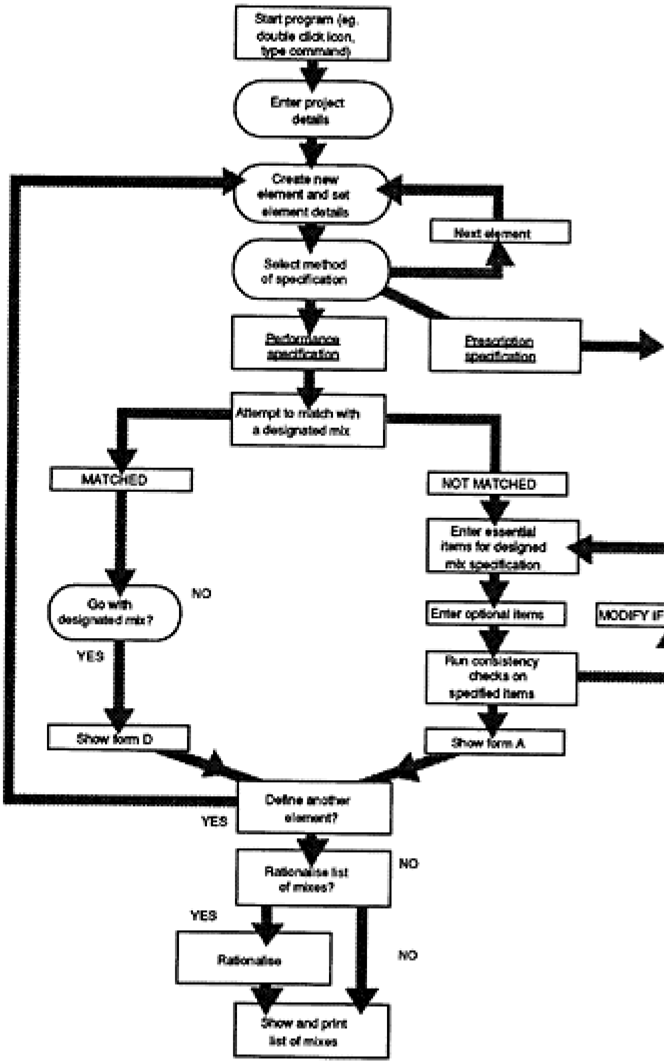

Figure 1—overview of program flow

- a portable graphical user interface—generally using a set of features that are common to all graphical operating systems but which will be presented on each platform with a 'native look and feel' ie conforming to the appropriate graphical user interface standard.
- an on-line help/reference guide—based around accepted technical literature (eg. *The Chemistry of Cement and Concrete* [5], Concrete Society Technical Reports and BRE Digests, etc.).

Knowledge base

The underlying philosophy for the knowledge base is traditional set theory and pattern matching. To facilitate development and to minimise demands on memory requirements, the knowledge base will be produced in a modular manner, consisting of a number of separate knowledge bases which will call each other as required. The knowledge base is being built using a set of rules that operate on user-defined and system objects (variables, strings, flags etc). Rules are developed in an 'If-Then-Else' format and as separate entities without a predefined processing route. This approach ensures that all applicable rules are processed and facilitates rapid modification to accord with changes in the Standards and Regulations.

The illustration in **Figure 2** suggests that each knowledge base is plane, with distinct boundaries interacting only in well defined areas where one overlaps the other, ie. the relationship between hardened properties and the placing requirements for construction providing constraints on the fresh properties of the concrete. However, the reality is somewhat less clear. Whilst each knowledge domain is well defined, the interaction between knowledge bases is largely dynamic and the knowledge base behaves more like a group of fuzzy, amorphous masses with both strong and loose links.

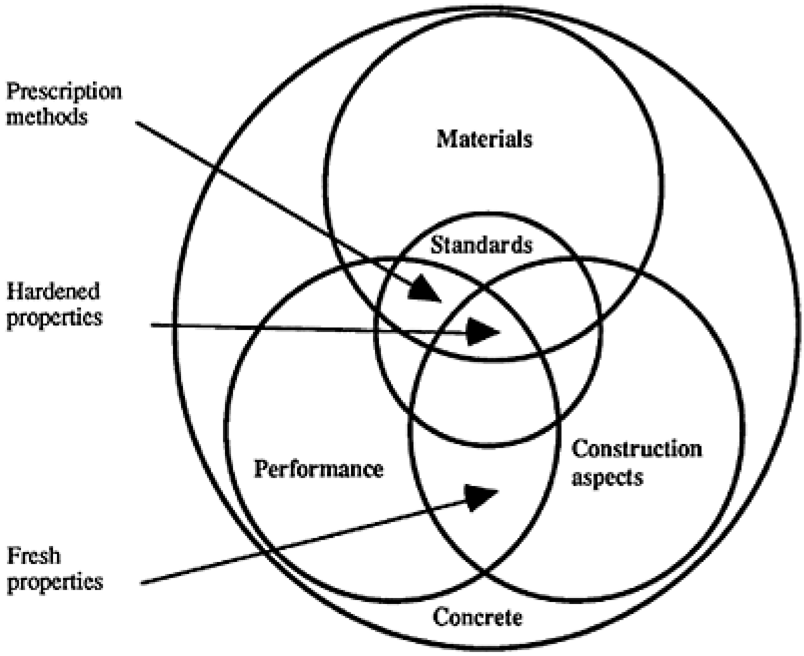

Figure 2—overview of system knowledge bases.

In addition to information contained in the Standards and Regulations, the knowledge base will also offer assistance in selecting criteria for non-typical end uses. For instance, selection of cements and aggregates for concrete exposed to industrial chemicals, high-performance concretes and architectural finishes. The information used to develop this part of the knowledge base will be based upon information that already has a proven history of satisfactory use, with the lead reference being the ACI Manual of concrete practice [6].

Once the most appropriate concrete has been selected for each element in a project, a rationalization process will be evoked. This routine will look for similarities and differences between concretes and reduce the number of different concretes used on a project to an economic minimum. The aim of this routine is to improve construction economy by providing flexibility for maximising the use of any given concrete.

User interface

A object-driven user interface will be provided, designed for rapid specification production for both individual concrete elements and projects containing many elements. A schematic of the interaction between the knowledge base and graphical user interface is given in **Figure 3**.

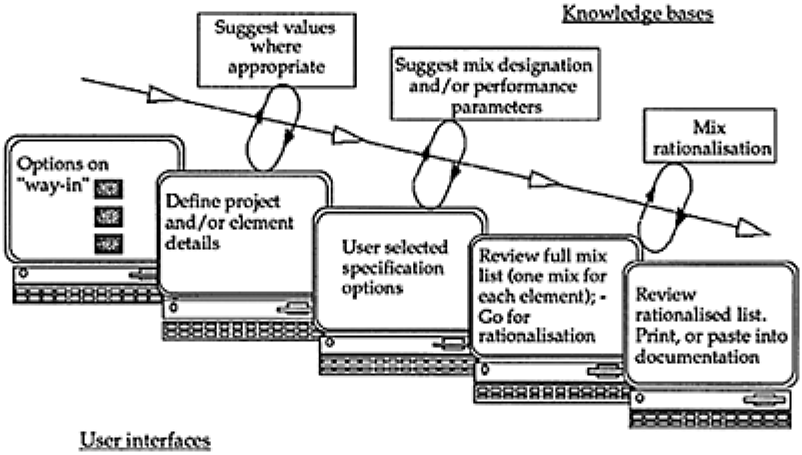

Figure 3—schematic of system process.

In general, the user will not be required to directly input information such as sulfate class and exposure conditions. It is intended that as much information of this nature as possible will be collected using indirect methods. Adoption of this methodology is intended to ensure accurate interpretation of all available data. It will also permit updates and new releases of standards and regulations to be supported without the requirement of re-engineering the program, new data files only will be required. Once administration data has been collected, the first dialogue routine involves a determination of the type of project/element being considered and will then suggest much of the environmental exposure conditions. For example, if the structure type is a road bridge in the UK, the system will assume that concrete will be exposed to chloride attack from de-icing salts, that it will be exposed to freeze/thaw action and that the Specification for Highway Works [4] and BS 5400 [7] will be the lead Regulation and Standard. Naturally, the user will be offered the opportunity to confirm or amend the suggested data but this approach will help to ensure that all items required to produce a correct specification will be provided, consistent with the rules of the appropriate Standard/Regulation.

Similarly, where site investigation information is available, the user will be asked to enter the details of the chemical analysis rather than just the sulfate class. This approach has two benefits; it will help to avoid interpretation errors and will ensure that the available information is interpreted in accordance with the current requirements.

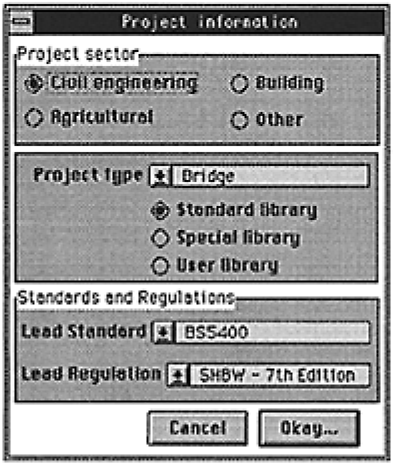

Figure 4—Current project information window in native Windows and Motif look.

Information relating to section size and envisaged placing method will be used as part of the rationalization process as well as for checking such items as limits on the heat of hydration of cement and suitability of aggregate size. Guidance on appropriateness of the envisaged placing and finishing methods together with workability selection will be given at this stage. Guidance information will be appended to the specification to be confirmed or adapted by the purchaser of the concrete (contractor).

Reference guide

The reference guide will provide interactive on-line help and operate as a stand-alone guide to the philosophy of concrete mix design, materials behaviour, concepts of Standards and Regulations, specification and site practice together with other items. Using text, diagrams and photographs the guide will contain background information on the concepts of concrete technology. Some topics that are difficult to describe using still images, for example cohesion, will also be described using video. However, this facility requires extensions to some operating systems that have yet to be developed and may not be available on all platforms.

Anticipated benefits

It is envisaged that this system will be a potent method of facilitating the rapid introduction of

The Contractors Interpretation

The specification and Bill of Quantities are poorly written, riddled with ambiguities and do not support each other. The specification section is lacking in information and relies heavily on the technical data sheet. A point worth emphasising is that eight (8No) pages of the document had been used for photographs yet the Bill of Quantities covered only one page.

When challenged on this point, most consultants will reply thus ‘A Competent Contractor should be aware of the underlying problems in tendering for a concrete repair project’.

It would probably be more reasonable to emphasise that after an inspection of contract documents in the mould of Table 1 and 1A, a competent contractor should return the documents to the consultant, diplomatically explaining that due to the lack of information it is not possible to submit an accurate bid. However, whilst most contractors would relish the thought, prevailing necessitous conditions in terms of work load, dictate otherwise.

This attitude is welcomed by the ‘sharp practising’ contractors surveyor who will recognise the ambiguities and grasp the opportunity to inform the consultant that ‘due to the absence of information the progress of the works is being delayed’, and subsequently informing him that the contractor will be seeking an extension of time under the relevant contractual clause. This will result in a claim being submitted, followed by an untidy, protracted and expensive period of letter writing possibly ending in litigation.

A senior partner of a leading U.K. construction consultancy firm revealed that in his experience, almost all of the contractual disputes thus claims have arisen as the result of insufficient pre-contract/pre-tender information. The reality of this point was endorsed by a front page article featured in an August edition of the ‘Construction News’, which stated ‘industry reels from self-inflicted wounds as litigation spirals upwards’. The same article continues to state that claims consultants, lawyers and arbitrators have never been busier and that litigation is presently estimated to cost the industry £3000 million a year. A percentage of this amount is allocated to the repair sector of the industry.

Alternatively a well produced tender document should be sufficiently informative to allow the contractor to submit a tender without having to guess the extent, nature and dimensions of the repairs on all elevations and levels.

This in turn will assist the contractor to assess the need for the provision of scaffolding (if required) and to estimate the time scale for carrying out the repairs. This point is emphasised by the inclusion of Table 2 and Table 2A which have also been extracted from an actual tender document.

the advances in concrete technology described in UK national and European standards to aid construction economies. Where adopted the program will enable more efficient use of materials and will make more efficient use of specialist resources (engineer, producer and contractor).

By reducing errors and ambiguity in concrete specifications it is predicted that the number of specification related disputes will fall significantly, together with an associated decrease in the amount of returned/waste concrete.

ACKNOWLEDGEMENTS

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Theme 3
VERSATILITY OF CONCRETE

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Leader Paper

THE VERSATILITY OF CONCRETE

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ABSTRACT. Concrete is the most intensively used man-made material and comes only second to water in volume consumption. Its success is due to: ingredients of world-wide availability at low prices, accessible technology, low energy consumption, feasibility of engineering its properties and performance to suit a wide variety of requirements. The range of properties that can be achieved with concrete is tremendous and is hardly matched by any other material: compressive strengths as low as 0.1 MPa and as high as 800 MPa are feasible; densities between 100 kg/m^3 and 5,000 kg/m^3 are also achievable.

The paper lists and discusses the enormous variety of applications where concrete is advantageously used. It is shown that in infrastructure applications all building materials have concrete as a competitor but that the reverse is not true. Based on its competitive advantages it is foreseen that concrete will continue to be the most versatile and used building material of the next century. To keep this leadership some of its shortcomings need to be addressed and alleviated.

Keywords: concrete versatility, tools, properties, applications, competitive materials

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INTRODUCTION

Concrete is the most widely used building material in the world today. We find concrete in all kind of residential, public and industrial buildings, in underground structures, in all transportation structures such as pavements, railroads, bridges and tunnels, in structures for water and sewage treatment, in power stations, in offshore structures, dams, etc. Most of the achievements of our modern civilisation depend on concrete, forming with steel the basis for modern structural engineering.

The total world production of concrete in 1995 lies around 10,000 tons, i.e. about 2 tons for every human being. To put this figure in perspective, if all the concrete produced in 1995 were used to build a conventional two-lane concrete pavement along the equator, the road will turn about 50 times around the globe.

The reasons for the wide usage of concrete are not difficult to find—the raw materials are in plentiful supply and available almost everywhere, requires low energy to be produced, costs relatively little compared to other building materials, a wide range of production and applications technologies is available and, last but not least, its great versatility in applications covers a wide range of demanding performance requirements.

Concrete competes with all major building materials—timber, steel, and other metals, asphalt, rock, plastic, glass, bricks, ceramics—because of its versatility in application.

The object of this presentation is to highlight the versatility of concrete. According to the definition, versatile means 'interested in and clever at many things'. We will delve into the tools at the disposal of the concrete technologist to achieve that versatility and the results that have so far been achieved in terms of properties and applications. Essentially, we will discuss the following questions:

how can we modify concrete properties?—versatility tools,
what results can we achieve?—versatile properties of concrete, and
for what purposes we do it?—concrete versatility in practice.

We will consider practically all concrete types and applications, whereby under concrete we understand a "composite material that consists of a binding medium within which are embedded particles or fragments of aggregate" (ACI-definition). The binding medium can be Portland cement, blended cement, cement mixes, mineral admixtures and chemical admixtures, special cements and water. It is evident that this rather broad definition covers a very wide range of materials and applications.

VERSATILITY TOOLS

One of the main tasks of a concrete technologist is to design concrete mixes suitable for an intended use. In fact, before designing the mix, he has to define the most convenient process to build the structure and specify the requirements of the mix accordingly.

Once the requirements of the concrete have been specified, the concrete technologist has to engineer the characteristics of the composite to achieve the expected performance at the lowest possible cost.

He has a limited, yet powerful, tool box to engineer the properties of concrete. He can act basically on four fronts:

♦ Rheology of fresh concrete,
♦ Chemical processes,
♦ Pore structure of the composite,
♦ Fracture mechanics (especially when fibres are added).

Let us look at them in some more detail.

Rheology of Fresh Concrete

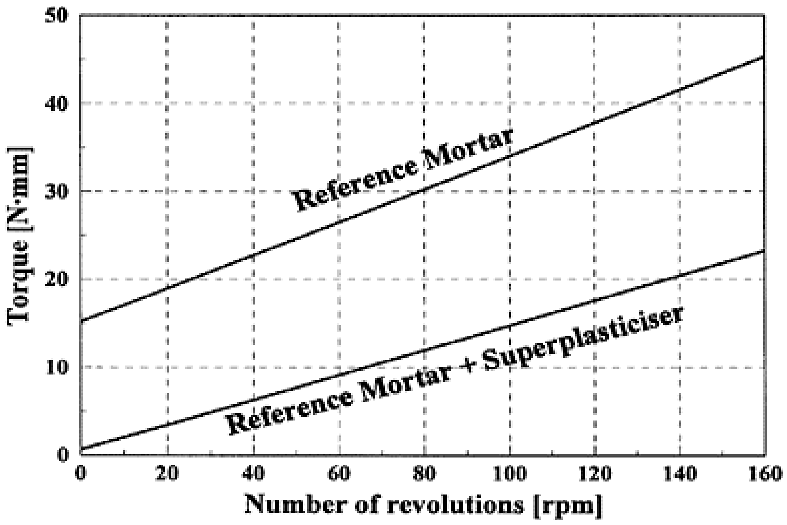
As is well known, the behaviour of fresh concrete and mortar can be compared to that of a Bingham solid, defined by two parameters, the yield stress and the plastic viscosity, which can be modified through mix design.

The main tools available are:

♦ the content of water in the mix,
♦ the amount and type of fines,
♦ the use of chemical admixtures,
♦ the grading of aggregates.

Figure 1 shows how the addition of a superplasticiser reduces the yield stress and the plastic viscosity, bringing the behaviour of the reference mortar (Bingham solid) close to that of a Newtonian liquid (yield stress=0).

Fig. 1—Rheological behaviour of mortars



By making good use of those tools we can produce concretes that, on the one hand, behave almost like liquids, i.e. they flow unaided, filling each and every available empty space in the forms, without segregation. On the other hand, we are able to produce concretes that can be demoulded immediately after casting (e.g. precast concrete blocks) or that are even able to withstand heavy loads in their fresh state (roller compacted concrete). Depending on its rheology, concrete can be transported by conveyor belts (up or down hill), flow freely through chutes or tremies, be pumped over long distances and heights or be projected through a nozzle (shotcrete).

Chemical Processes

The main chemical reactions that take place in a concrete system are those associated with the hydration of Portland cement; they start as soon as cement is put in contact with water and continue for years under suitable hydro-thermal conditions.

Both the intensity and the kinetics of the chemical reactions taking place within the concrete system can be controlled. The tools available are:

- ♦ cement composition,
- ♦ mineral additions added to the cement or into the concrete mixer,
- ♦ fineness of the cement,
- ♦ cement content of the mix,
- ♦ use of chemical admixtures, mainly retarders and accelerators,
- ♦ thermal treatment.

By proper use of these tools it is possible to achieve concretes that set almost instantaneously to concretes that take hours and even days to set as is the case of the long-life mortars. Also concretes that gain strength very rapidly like high-early strength cement concrete or steam-cured concrete or very slowly, e.g. blast furnace slag cement concrete. Concretes that develop a high or low amount of heat of hydration can be designed, e.g. for winter concreting and mass concrete, respectively.

By adding certain chemicals it is possible to generate and entrap gases in the mass (frost resistant concrete and aerated concrete) or to promote certain controlled expansive reactions to compensate for the drying shrinkage (expansive cements).

Pore Structure

Concrete is a porous material containing a system of pores covering a wide range of pore sizes.

Pores can be found within the aggregate particles, but also between the particles if the intergranular voids are not completely filled with cement paste.

The paste itself has a high porosity and at least three different kinds of pores can be found:

- ♦ Gel pores, in a size not much larger than a few times the size of a water molecule, i.e. 1 to 10 nm. These pores are responsible for most of the volume changes of concrete associated with gain or loss of water (drying shrinkage, swelling and part of the

creep). They are a structural feature of the hydration products and, as such, cannot be eliminated.
♦ Capillary pores, resulting from the space originally occupied by the water of the mix that has not been filled with hydration products. For good quality concrete the size of these pores is in the range 10–50 nm. For low quality concrete it can reach 10,000 nm (i.e. 10 µm).
♦ Air micro-bubbles, originating from the use of an air entraining or foaming agent. These bubbles are typically in the size range of 10–100 µm.

The main tools to modify the pore structure of concrete are:

♦ water/cement ratio (volume and size of capillary pores)
♦ cement content (volume of gel and capillary pores)
♦ use of micro-fillers (e.g. silica fume)
♦ use of air entrainers (volume and size of micro-bubbles)
♦ use of porous aggregates
♦ use of special grading of aggregates (drain concrete).

By proper engineering of the pore structure of concrete we can modify at will (naturally between certain limits) properties such as density, strength, modulus of elasticity, thermal insulation, permeability, radiation absorption, etc.

Fracture Mechanics

One of the characteristics of concrete more difficult to modify significantly is one of its inherent shortcomings: its brittleness or low fracture energy and its low tensile strength.

A remedy to overcome this limitation has not yet been found within the traditional range of component materials of concrete. However, one way at least to palliate this problem has been found through the use of fibres as a further concrete component. Properly designed concrete containing the right volume of the right type of fibres can increase its ductility dramatically. These improvements have not been sufficient to avoid the use of steel reinforcement for structural concrete.

Nevertheless, this is a field where developments are expected in the future, particularly as fibres of more sophisticated materials become available at relatively lower prices. For special applications this type of high-technology concrete is produced already today.

THE ‘VERSATILE’ PROPERTIES OF CONCRETE

Table I presents the range of properties that can be achieved by a clever use of the tools available, as discussed in the previous Sections.

Figs. 2 represents the range of properties achievable for different concretes compared with those corresponding to several materials used in construction (construction steel, aluminium, timber, plastics and glass).

It can be seen that concrete is the most versatile material of the six in terms of density (together with plastics), E-Modulus and thermal conductivity. Its limited performance in terms of tensile strength (when unreinforced) is evident as well.

CONCRETE VERSATILITY IN PRACTICE

In the previous section we have discussed the wide range of properties that can be achieved by proper use of the tools at our disposal.

By using the know-how available we are able to produce a complete palette of products designed to suit specific needs of the construction industry. The extensive usage of the great number of concrete types in the various constructions using a variety of production methods is shown in Table II, illustrating the extraordinary versatility of concrete. None of the competitive building materials reaches such a wide usage in man-made structures.

It is of interest to show the building materials which are competitive to concrete in the different fields of application. Table III presents the situation for some typical cases.

Table I Typical values of important properties for different types of concrete

Property	**Typical Values**	**Concrete Type**
Density (kg/m^3)	200–800	Lightweight (Insulating)
	800–1400	Lightweight (Insulating and Structural)
	1400–2000	Lightweight (Structural)
	2000–2600	Normal weight
	2600–5000	Heavyweight (Radiation shield)
Compressive Strength (MPa)	0.5–2.0	Flowing Fills
	0.4–30	Lightweight
	15–60	Normal Strength
	60–150	High-Strength
Modulus of Elasticity (GPa)	2–15	Lightweight
	15–35	Normal Strength
	30–50	High-Strength
Thermal Conductivity (W/m.K)	0.1–1.0	Lightweight
	1.0–2.5	Normal weight
Permeability to Air (10^{-16} m^2)	<0.1	High-Strength (>65 MPa)
	0.1–0.5	Good Quality
	0.5–2.5	Medium Quality

	2.5–12.5	Poor Quality
	>12.5	Very Poor Quality
	~∞	Draining, aerated

Fig. 2—Typical properties of construction materials

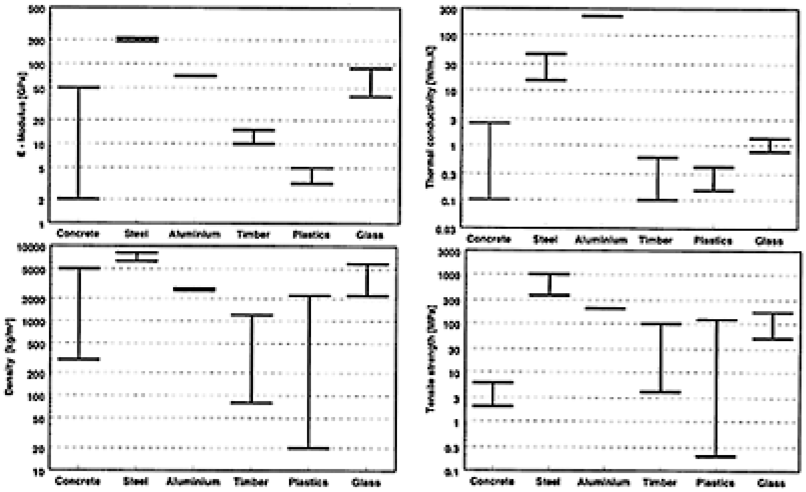


Table II Production methods, types of concrete and structures

Production methods	Concrete types
Ready-mixed concrete	Plain concrete
Precast concrete	Reinforced concrete
Site mixed concrete	Pre-stressed concrete
Shotcrete	Light weight concrete
Roller compacted concrete	High performance concrete
Pumped concrete	Fibre reinforced concrete
Slipforming	Mass concrete
Underwater concrete	Mortars
Vacuum concrete	Repair mortar and concrete
Steam curing	Flowing fill
Autoclaved concrete	Heat resistant concrete
	Concrete for aggressive environment

	Coloured concrete
Structure types	
Buildings (residential, public, industrial, slabs, columns, decks, floors, partition walls, fundaments, silos, pools)	
Transportation structures (rail or roads, roads, tunnels, bridges, pipelines)	
Sanitary structures (water and sewage treatment)	
Offshore structures	
Military structures	
Hydro structures (dams. water ducts, embankments)	
Power stations	
Hazardous waste containments	

Table III—Concrete vs Competitive Materials

Application	**Main Competitive Materials**	**Position of Concrete**
Buildings Frame columns and bearing;	steel, timber masonry	strong
walls beams, slabs, etc.	steel, timber	strong
Roads	asphalt, soil	weak
Bridges	steel, timber	strong
Dams	soil	strong
Partition walls	bricks, gypsum	weak
Pipelines	steel, plastics	balanced
Roofs	ceramic tiles, steel	weak
Tunnels	steel, natural stone	strong
Skyscrapers	steel	growing
Offshore structures	steel	strong
Foundations	masonry	strong
Sanitary Structures	plastic, ceramics	balanced
Hazardous wastes containment	steel	strong

Table IV presents some of the many different types of concrete, the tools used to achieve their specific characteristics required and various applications.

Table IV—Different types of concrete, characteristics and applications

Designation	Tools Used (*)	Characteristics	Main Applications
Flowable Fill	1, 2	Self-compaction, low-shrinkage, controlled low strength (<0.5 MPa).	Backfill of excavations
Permeable concrete	2	Monogranular aggregate, low cement content (100–150 Kg/m3), moderate strength (5 to 30 MPa), moist-earth consistency	Draining pipes, slabs
Impermeable concrete	2	Low w/c ratio (<0.45)	Reservoirs, petrol stations pavements
Foamed concrete Aerated (autoclaved) concrete	1, 2, 3	Air bubbles (0.2–1 mm) occupy 30–80% of total volume; w/c=0.5–0.6; dry density: 150–1550 kg/m3, f'c: 0.5–10 MPa, E: 1–12 GPa, λ: 0.1–0.7 W/mK, high shrinkage (when not autoclaved), good frost resistance, high carbonation rate	Thermal insulation of buildings, swimming pools, etc. Filling of excavations and out of order reservoirs. Artificial sport grounds, structural elements and masonry
Roller Compacted Concrete	1, 2, 3	Dry consistency, compaction by several passes of heavy rollers. Composition according to use: —Low to moderate cement content —Normal cement content	Dams Roads
Frost Resistant	2, 3	Entrained air, right amount and size of bubbles. Sometimes rapid hardening. Frost resistant aggregates. Low permeability.	Cold climates Cryo tanks
Underwater	1	Soft or flowing consistency, highly cohesive often through special admixtures	Place concrete under water table
Mass Concrete	3	Low heat generation to avoid thermal cracking	Dams, massive foundations, thick elements
Shotcrete	1, 3	Workability and cohesion suitable for projection with minimum rebound, fast stiffening	Embankments, Tunnelling, Mining repair, rehabilitation
Mortars	1, 2	Workability, cohesion and water retention. Low shrinkage and relatively low strength. Addition of lime, fillers and air entrainers	Masonry, renderings, repair
Coloured concrete and	3	Use of white cement and/or suitable pigments	Architectural purposes reflective elements

mortars			
Pumped concrete	1, 3	Workability and cohesion suitable for pumping, retarders to delay setting	All kind of concrete constructions
Precast concrete	1, 2, 3	Normally stiff mixes, fast setting and high early strength. Usually thermal curing	Concrete elements, Pre-stressed bridges
Concrete for aggressive environments	2, 3	Low permeability (low w/c ratio), components that are resistant to the aggressive media, surface treatments	Structures exposed to aggressive media
Fibre concrete	4	Fibres (steel, synthetic, glass) are added to improve the cracking resistance, durability	Industrial floors, impermeability, impact, explosions
High performance concrete	4	Low w/c ratio, silica fume or other fine silicas, special aggregates, fibres	High performance structural members, wear parts, machine parts
(*) 1 : Rheology of fresh concrete, 2: Chemical Processes, 3: Pore Structure, 4: Fracture Mechanics			

Two extreme examples should illustrate and complement the information given in table IV.:

Flowable Fill

This is a material intended to be used to backfill excavations (for example to fill the gaps between the excavated soil and pipelines, etc.). Traditionally this have been done by placing and compacting granular materials in layers. In many cases the material was dumped full depth into the trench, never tamped and therefore not adequately compacted.

The idea was to replace that traditional technique with another, allowing fast filling of all voids, easy re-excavation and fast re-opening of the road.

The properties required are:

- ♦ self-levelling (i.e. no compaction should be required),
- ♦ no shrinkage and
- ♦ very low strength (sufficient to support the construction of a pavement on top but low enough to ensure easy re-excavation when required).

The recipe of this kind of material varies, but as a guideline the following composition can do the job.

Portland Cement	25 kg/m^3
Water	225 kg/m^3
Sand	1105 kg/m^3
14 mm Stone	560 kg/m^3
20 mm Stone	387 kg/m^3

Supplementary cementitious materials such as fly ash, slag, etc. can also be used.

Strengths not exceeding 0.5 MPa at 28 days are achieved, mostly through expulsion of mixing water (cement acts mainly as a lubricant) and self-compacting. The low amount of cement guarantees the non-shrinking behaviour.

High-Strength (High Performance) Concrete

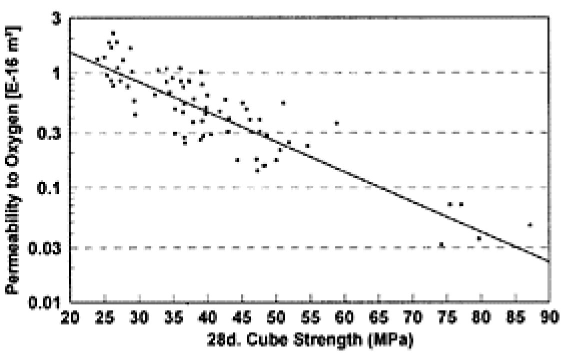
This is simply an extension of the conventional concretes, that became possible as a consequence of the development of high performance water reducers.

Generally speaking, these concretes present a w/c ratio between 0.40 and 0.28 (values as low as 0.16 have been reported), with compressive strengths ranging from, say, 60 to more than 150 MPa.

To achieve the high strength, the volume and size of the capillary pores must be reduced as much as possible, by means of a low w/c ratio. Use of a high performance water reducer is thus mandatory. For strengths beyond ~100 MPa, silica fume is also required to improve the cement paste-aggregate bond and to act as a micro-filler. In all cases, aggregates of good quality and limited maximum size have to be used. Sometimes, supplementary cementitious materials such as fly-ash and/or slag are added as partial substitution of cement, which also helps in keeping the heat development under control.

Other properties are also associated to this highly dense system, such as low permeability and the subsequent higher resistance to the ingress of aggressive substances. Fig. 3 shows the effect of raising the strength on the air-permeability of concrete.

Fig. 3—Relationship between Permeability and Strength of Concrete



The recent developments in the area of ultra high strength cementitious systems al-lows to produce materials having properties approaching those of steel and other metals or ceramics. One example are the macro defect free materials (MDF).

The Reactive Powder Concrete using a special concrete mix design with Portland cement, silica fume or other fine silicas, superplasticiser, well graded fine sand and steel fibres can be used for high performance structural membranes. The Reactive Powder

Concrete exhibits ultra high strength up to 800 MPa and high ductility at the same time. Another high performance material used in security products (safes), wear components, machine parts and other structural application is Densit, based on Portland cement, silica fume and superplasticiser.

FINAL REMARKS

At the end of this tour, that took us around the present of concrete as the most versatile and widely used building material of modern times, we should have a look into the future. Will concrete still have such a predominant role in the future? Is this predominance assured for the years to come?

The answer is difficult. History proves that no single human development remains predominant forever and, due to depletion of resources coupled to technological development, it is possible that concrete will be superseded in the long-term by another material or other materials which could possibly be unknown today.

However, the raw material reserves for making concrete are assured for many years to come and it is unlikely that some other material will take over in the foreseeable medium to long-term.

Moreover the full potential of concrete as a structural material has not yet be realised. The obvious drawbacks of concrete such as low ductility, low tensile strength, limited durability in certain situations, poor aesthetic and environmental image constitute a challenge for the concrete specialists to find solutions and so even increase the share of concrete on the building materials market. In particular the further reduction of CO_2-emission and energy consumption in the production of cement and concrete, recycling of industrial by-products and aggregates form demolished concrete shall help to make concrete more competitive and versatile. Overcoming the apparent contradiction between the low cost image and the high performance potential is another challenge for the concrete community.

Normal concrete will continue to be the major building material for quite some while. High technology concretes, as they can be produced today need much more stringent quality control and their application will always be based on the development of more sophisticated engineering projects.

Concrete is even considered as the possible material for construction in the outer space for the same reasons it has found such acceptance on earth.

We can confidently state that none of us here today will witness during our lives the replacement of concrete as the most versatile building material.

CONCRETE MASONRY: A DIFFERENT DIMENSION FOR HOUSING IN DEVELOPING COUNTRIES. THE 40 YEARS EXPERIENCE IN COLOMBIA

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ABSTRACT. The use of Concrete Masonry (CM) for low-cost housing in Colombia has been determined by local physical, technical, economic and social conditions, not different from those existing in other places but, maybe, in different combinations. Even though it is difficult to talk about special achievement in CM, it has been possible to define guidelines for the physical characteristics of low-cost housing and for the construction with Concrete Blocks (CB). A few changes have been made to the dimensions of CB used in several projects, and there is a defined structure for on-site production of standard and special blocks, both in urban as in rural environments. It allows great savings due to the input of free labor (in social-interest programs), as part of the payment for each house, giving CM a special social or well-being significance, further than providing the wall or the dwelling itself. This paper is based on Reference [1].

Keywords: Colombia, Concrete blocks, Costs, Economics, Low-cost housing, Masonry, Self-help projects.

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INTRODUCTION

Governmental low-cost housing programs, started in Colombia during the late 50s, became of a capital importance during the early 80s, due to the persisting immigration to cities from rural areas. This phenomenon, originated in the political violence of the early 50s and in the historical social imbalance, made Colombia a very “urban” country (the ratio of urban to rural population (in %) changed from 43/57 in 1951 to 65/35 in 1985 [2]), even though the country has been agriculture dependent (coffee, flowers, bananas) until recently; so most of the information given in this paper deals with urban housing projects.

CB is the second wall material in Colombia after fired-clay units (bricks and blocks). There is no industrialized construction of wooden houses, and there is almost no production of panels for partitions, just some fiber-cement and concrete panels for farm houses or cabins (people consider a house to be strong only if it is made with masonry or thick walls). The use of earth walls is almost forgotten, except in distant, underdeveloped areas or for landscape purposes. Steel is just starting to appear in frames for buildings (there is the production of reinforcing steel, small structural sections and laminated-steel roofing). Clay is not available everywhere, or with quality enough to produce units for load-bearing and reinforced masonry, like in Colombia’s Atlantic coast, where CM is almost the only possible or available material for walls and partitions.

In most of the country (specially in large cities), competition is strong among clay and concrete; but clay masonry is more widely known and accepted by construction workers and users, since it is easy to handle (light weight, easy to cut) and very convenient to produce a good result with minimum technical knowledge. It makes that, unless for economic reasons (and recently due to the reinforcing requirements to attend earthquakes), clay masonry be the first-choice for new houses or for additions to old ones. In rural areas, houses are built with the available material; but a point in favor or CM is the possibility to be produced in small-scale plants, something technically impossible for fired-clay units. This paper presents the scenario of low-cost CM in Colombia under its main physical, technological, economic and social characteristics.

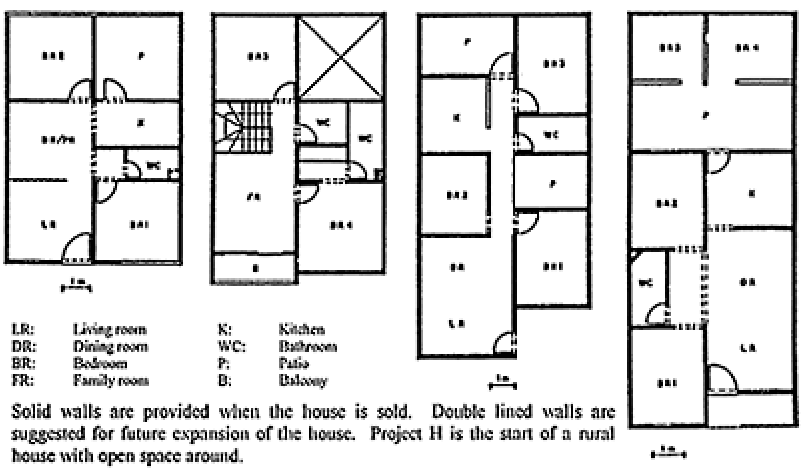
PHYSICAL CHARACTERISTICS

Typologies

Urban, One to Three-floor Houses

CM houses do not differ from one or two-floor, clay-masonry houses, built on load-bearing walls, to incorporate one or two dwellings. Very recently, developers introduced three-floor houses, to get more from scarce and expensive land, and as a solution between houses and low-rise apartment buildings. It makes low-cost housing to have defined typologies: 1 floor/1 dwelling (Figures 5, 7); 2 floors/1 or 2 dwellings (Figures 4, 13); 3 floors/1 to 3 dwellings (1 dwelling in 3 floors; 1 in first floor, 1 in the second and third; 1 in each floor) (Figure 16).

When a typology is defined on the design board, it does not mean the house is going to stay like it. Low-cost housing grows in every possible way, making the designed unit just the beginning. In one floor houses, the fiber-cement or clay-tile roof is changed for a concrete slab, in order to build a second floor and, maybe, a third one for rent. Something similar happens when part of the terrain is left for future growth or when, in two and three-floor houses, a part of the slab is left to build new rooms (Figures 2, 3, 9). The possibility "to build on top" makes a great difference when you buy a one or two-floor house or one in the three-floor units. The "owning of the air" is limited or restricted in two and three floor units. Those small dwellings, unable or difficult to grow, are called "apart/houses", even though they one house on top of the other.



Solid walls are provided when the house is sold. Double lined walls are suggested for future expansion of the house. Project H is the start of a rural house with open space around.

In Project E to G, E is the one-floor house as sold; F is the 1st. floor of the two-floor house or the modification when expanded; G is the 2nd. floor of the two-floor house or the modification when expanded. Urban projects only have a backyard as area for expansion (projects A, D).

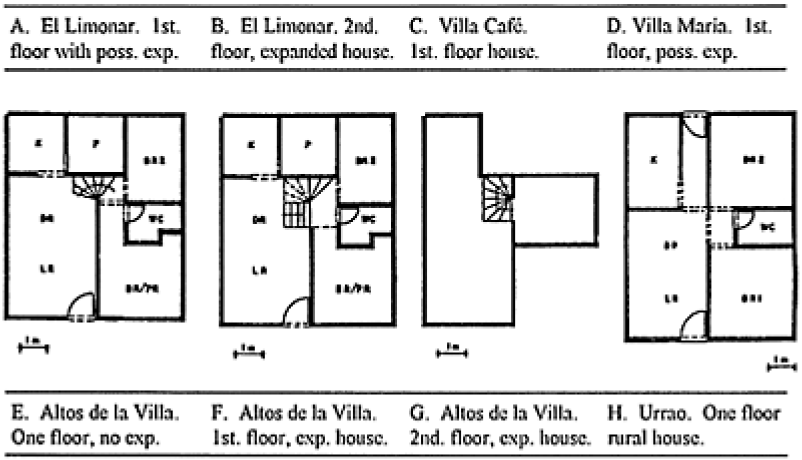


Figure 1. Layouts for different low-cost housing projects.

Even though it is necessary to get a permit from the city planning office for any modification, the requirements are often violated, with an intricate mix of materials and structural types. It could be found that over CM, clay masonry or concrete walls first floors, second and third floors are made with concrete or clay masonry, or even with a lightweight panel house, creating a dangerous situation in seismic regions, in steep-sloped terrain and in technically poor environments. These violations and the non-reported modifications are such that the yearly figure for the non-registered area for new or remodeled construction is between 35 and 100% of the registered one, depending of the controls by local authorities, with an average around 55% [3].

This is a delicate topic from the point of view of planning and safety. Responsible promoters design according with planning regulations and make it clear to potential owners, the restrictions for future expansions. But, once the houses are sold, those requirements are forgotten, and after ten years, it is difficult to recognize the original houses in a block (Figures 2, 3). It is advisable to be preventive, studying the local traditions in order to design with enough structural capacity to be used in the future, since planning offices are not capable to control every construction.

As an example, some of the planning regulations, in Medellín, for one, two and three dwelling low-cost housing are: Minimum areas for the lot, 54, 60 and 72 m^2, respectively, with frontal lot widths of 4,5 m for the first one and 6 m for the second and third ones. The minimum "embryonic" unit is 18 m^2, and should have one multiple space (living-dining-bedroom) a kitchenette and a bathroom (Figures 1A, 7). The minimum "regular" unit is the embryonic one with one or two bedrooms. The minimum area possible to be built must be 40 m^2 (for second or third-floor projects, a finished slab must be provided additionally to the embryonic area [4]).

Complete houses or apartments have united living and dining rooms, connected to the kitchen; a laundry area (exterior but covered in patios or terraces); one to three bedrooms and one or two bathrooms (sink, toilet and shower). First-floor houses always have a patio. Second and third floor houses have a terraces. One dwelling three-floor houses are sold with one unfinished floor (bare slab and walls) or with the possibility for the construction of one or two rooms (Figure 1).

Two indexes are used to evaluate the size of inner spaces and the space occupied by walls [5], and to calculate the consumption of masonry. For houses the average are: 0,598 m/m^2 (wall length/habitable (net) area) and 0,077 m^2/m^2 (area occupied by walls/net area). The second index divided by the first one gives the average wall thickness, in this case 130 mm. Values have been found from 0,512 to 0,666 m/m^2 for the first index and from 0,059 to 0,092 m^2/m^2 for the second one, with average wall thickness from 115 to 150 mm.

Rural Houses

It is uncommon to find rural-housing programs, except for reconstruction after natural disasters. Those programs, executed by responsible, mostly non-governmental institutions (NGOs), tend to be more complete, in services, than those developed for commercial purpose in urban areas. A church, a school, a soccer field or a concrete multiple-sport slab, a health center, a communal building and some workshops or commercial spaces are often provided. It is intended that typologies do not deviate form traditional ones, but have better services (Figures 1H, 5).

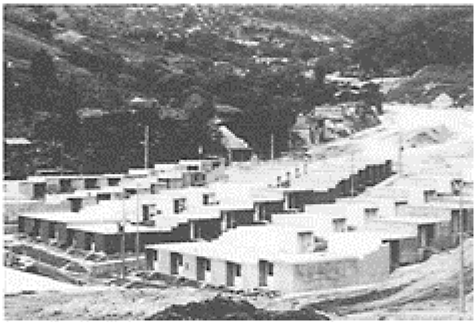


Figure 2. Low-income one-floor housing project (concrete walls), as built. Medellín, Damato, 1984.

Figure 3. Same neighborhood as in Figure 2, 11 years later (1994). A few one-floor houses remain.

Figure 4. “Centeneario Lasallista”. Four houses in one unit, Corporación Antioquia Presente, 1988.

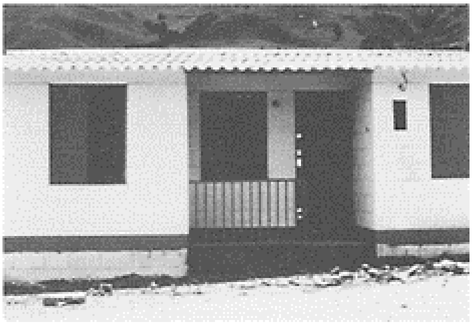


Figure 5. “Villa María”. Typical rural house built by Corporación Antioquia Presente, 1991.

Figure 6. “El Limonar”. Project built to move those living in high-risk areas. Medellín, Corvide, 1994.

Figure 7. “El Limonar”. One-floor dwelling (Figure 1A); job to be done in front before occupation.

Figure 8. “El Limonar”. View of a walking path with two sidewalks and central garden.

Figure 9. “El Limonar”. Modified houses (Figure 1B), some years after occupation.

Low-rise Apartment Buildings

It is usual for buildings to have two apartments per floor and stair shaft, and four to nine floors, as long as it is not necessary to go more than four floors up or down from access level, to avoid the use of an elevator required for more than five floors. This is done by arranging the buildings on sloped terrain and using ramps or bridges (Figure 17). The five story “multifamily” building is the most popular format for urban projects in the country, for upper low-cost and for middle-cost housing, no matter the structural system (concrete frames, concrete, clay or concrete masonry bearing walls or large concrete panels). Apartments go from 65 to 90 m^2 and the foretold indexes are 0,615 m/m^2 and 0,088 m^2/m^2, with an average wall thickness of 143 mm.

Urban Design

Regulations for urban design (area/dwelling, densities, distance to streets, constructions and streams, road or walking-path design) differ a lot from city to city. In Medellín, for low-cost projects, the main road section (bus route and emergency access) is 10 m (a 7 m pavement, two 1,5 m sidewalks), non including an optional frontal green area. Most of the houses are built in front of 6 m wide walking paths, with a 2 m central sidewalk and two 2 m green areas; or two 1 m sidewalks and one 4 m wide green area [4] (Figure 8). The walking paths should not be more than 150 m long when connecting two access roads or a road and a green area (at least 500 m^2). In both cases they should be cut by another walking path to reduce the block length to 75 m. Walking paths and sidewalks are a continues surface for slopes up to a 16 % or should have steps if slope is higher [4]. The slope is limited just by feasibility (Figure 13).

TECHNICAL CHARACTERISTICS

Several factors affect the technical characteristics of low-cost houses and buildings, generating a considerable amount of changes from original fast-thought designs:

Profile of the Terrain

Most of the Colombian cities are located on the Andes, meaning that a large amount of houses are built on slopes as steep as 25 to 50 % along walking paths. The only possibility to accomplish it is to design houses with the longest dimension perpendicular to the slope, making necessary to create terraces to accommodate every house or group of houses (Figures 12, 15). With time it has evolved to make the terracing for just half a house, since a total terracing requires to move large amounts of earth (more problems with water tables), and to build higher retaining or cantilever walls and a slab to make the floor for half a house. It demands a more strict regulation for the underground space not to be used since it is potentially accessible.

The second option implies to design a split house, with two longitudinal halves separated half a floor height, reducing excavations, allowing to have low retaining walls and less conflicts with water table. This type of accommodation reduces the conflict with the sidewalk and allows a more versatile design when a part of the house is left without the intermediate slab for future growth. In this scenario, CM allows all the structure to be built with the same material (CB) but different reinforcement conditions, improving construction speed and esthetics.

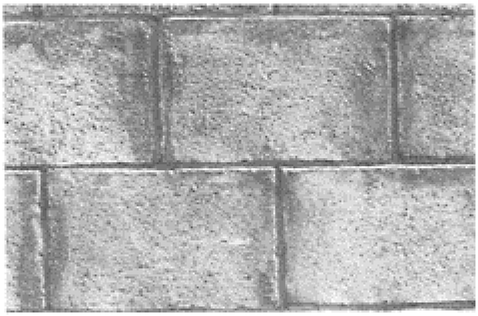


Figure 10. 250×400 mm “tall blocks” used for low to medium cost housing

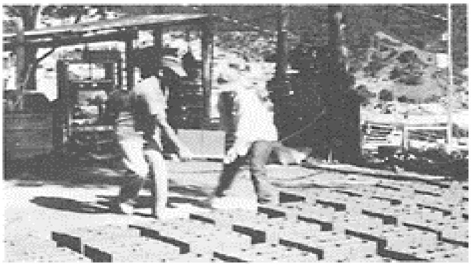


Figure 11. “Altos de la Villa”. On-site production of concrete blocks. Medellín, PSI, 1993.

Figure 12. “Altos de la Villa”. Terracing for split-floor houses (Figure 1E), made with “tall” blocks.

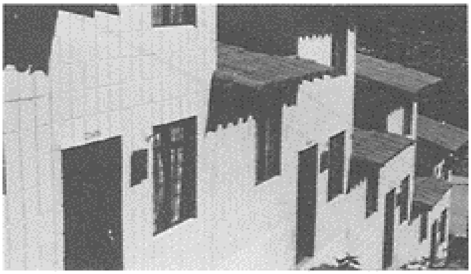


Figure 13. “Altos de la Villa”. The two-floor house as provided; expansion in half the second floor.

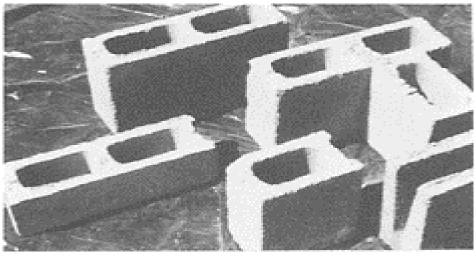


Figure 14. “Terracota”. Five on-site made CB types used to build the houses. Medellín, PSI, 1994.

Figure 15. “Quintas de El Salvador”. Distribution for one and two-floor houses in two or three floors.

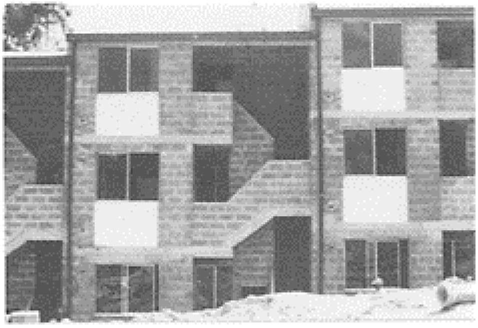


Figure 16. “Portón del Limonar”. 3-floor, 3-family housing units. Medellín, Corvide, 1994.

Figure 17. “Nueva Andalucía”. 5-floor apartment buildings. Medellín, Bravo-Restrepo, 1978.

Seismic and Structural Codes

The Andes is a active seismic region. The Colombian Code on Seismic-Resistant Structures (CCCSR-84) has Title D dedicated to "Structural Masonry" and Title E to "One and Two-floor Buildings" [6]. Non-reinforced load-bearing masonry almost disappeared. The Code ask for architectural blueprints and structural calculations to be sent to the City Planning Office to get the construction license. It works well for authorized construction, not for the unauthorized one. In order ease the consequences of the lack of total control in cities, efforts are made [7,8] to disseminate information for foremen on construction of one and two-floor houses. In low-cost CM houses there is less pressure for the removal of walls than in apartments. Clay masonry is the usual material for facades. Vertically-perforated clay block appeared some years ago but they are not available everywhere. CB are almost everywhere (although not in the best quality), providing the vertical cells for reinforcement purposes, reducing costs and labor.

Block Types

Most of low-cost CM housing is built with standard 400×200 mm blocks, half and "U" units, very seldom using architectural ones. The Structural Code allows 125 mm wide blocks for load bearing purposes; 150 mm ones are common; 100 mm units are used for partitions. Something new is the use of 250 mm tall units; preliminary results indicate that there is an improvement in strength of the masonry prisms (Figure 10). This change has gained acceptance since: There is no additional effort during production, reduce one layer of mortar per meter in height and, since block laying is mostly paid by units, there is a reduction of 20% in labor costs and some savings in mortars, more expensive than blocks (Figures 10, 11, 13). Some contractors producing block on-site, including special units like "U", "T", mid-height and tall blocks (Figure 14).

ECONOMIC CHARACTERISTICS

Costs Structure

Commercial construction

Official policies determined that all construction material for housing is excepted from the 14% Added Value Tax (AVT), and there has been a subsidy for low-cost housing. In 1939 the Territorial Credit Institute—ICT was created to build low to medium cost housing. Because of an accumulative deficit due to unpaid quotas, bureaucracy and complains for bad quality, ICT was liquidated and the INURBE began providing a direct subsidy in 1992. The subsidy has three scales depending on the population (Table 1). The top price of a dwelling must be US $ 12 590, 15 108 or 16 997 respectively, to be eligible for the subsidy, being it equivalent to 100, 120 and 135 Minimum Monthly Salaries—MMS. The subsidy is given as a part of the down payment. The rest is paid in 15 or 20 years in Constant-Acquisitive-Power-Units—UPACs. The units increase their value according to inflation. In most cases, subsidies are higher if given through a Social Compensation Box, a system to benefit the lower-paid people, with founds coming from a 4% of all the salaries.

For a builder of low-income CM housing, the cost structures is something like: Land 17%, construction 54%, financial 6%, indirect 13% (design honorariums, promotion and sales, le-gal, registration and insurance), utility 10% for a retail price of some US $ 225/m^2, of which the under-structure and services costs are US $ 45/m^2.

Social programs construction

In programs like for relocation, there are special considerations, depending the way they are structured. The approach of the Corporation Antioquia Presente—CAP, an institution created to channel the donations made in the Department (state) of Antioquia after disasters, is to have the self-help or self-construction component both in urban and rural programs. Each family must have a person (at least 15 years old), working full time (some 1 200 hours, 8 months). Additionally it is required to pay the legal fees, to gather US $ 1 220 through savings in one of the UP AC system corporations, and to apply for the INURBE subsidy (Figure 5). This type of house has a direct cost for the CAP between US $ 85 and 100/m^2 plus the costs of the communal components. The part of the cost not paid in labor, money or subsidy, is obtained by the CAP from the donation campaigns and from governmental funds or international cooperation.

The key point in this type of programs is the CAP self-construction scheme. For each program they set-up a group made by a technician in construction, a foreman, a social worker and the representatives of each family (plain labor). After the required time, the representatives learn enough to work in construction and some of them have conformed pre-cooperative groups to contract with private construction companies. 60% of this labor is made by women.

El Limonar, a recent program developed, in Mcdellin, by Corvide, a corporation under the Mayor’s Office, build 25 m^2 houses on 54 m^2 lots, with a price of US $ 3 655, to accommodate people from areas prompt to land slides (Figures 6 to 8). The City recognizes US $ 2 070 for the old dwelling (to be demolished) and the owners pay US $ 80 as down payment. Once they are registered in the program, INURBE gives a subsidy of US $ 1 220, leaving only US $ 285 to be paid in 15 years with an interest rate below the commercial one.

On-site production of blocks

This topic generates discussions among block producers and contractors. The first ones try to keep the domain of the activity, arguing professionalism, exclusive dedication, quality, full employment, etc. Contractors think they get economics since most of the costs are equal for both in urban areas, but contractors do not pay the transportation to the site. On-site small plants can produce standard blocks, almost any shape, being quality more a question of responsibility, and the units adequate for reinforced structural CM. In both schemes, block quality is controlled by the Colombian standards, very much like ASTM’s. The concrete composition do not differ from the ones used anywhere else. The way labor and production handled is argued by producers to be unfair, saying that most of the labor on the site do not get full salaries and benefits. They also argue pollution and tax disadvantages of on-site production, but it is equivalent to make concrete on-site, instead to buy ready-mixed concrete. Arguments are endless. Contractors report to make blocks for 80% the factor price and to have the additional savings of the hauling, equivalent to near 22% of this amount; so the difference is near 30%. The factory prices before AVT for 200×400 mm blocks are: 0,31; 0,40; 0,48 and 0,63 US Dollars for 100, 120, 150 and 200 mm wide units. Those figures look attractive but beware of the hidden costs contractor usually do not consider, the relative costs of materials, hauling, labor, taxes, planning and environmental restrictions, trade policies, available on-site space, adequate down-scaled and cheap equipment and molds, etc.

CONCLUSIONS

The Colombian experience demonstrates that concrete masonry is not only an alternative but a very versatile, economical and technically efficient system to build low-income housing in developing countries that should be studied for the benefit of the population.

Table 1. Subsidies for low-cost hosing in Colombia (US Dollars) (as August, 1995)

INSTITUTION	CONSTRUCTION	TOWN POPULATION, thousands		
		<100	100 to 500	>500
INURBE	IMPROVEMENT	1638 (210*)	1 638 (210*)	1 638 (210*)
	NEW	1 638 (210*)	1 950 (250*)	2 340 (300*)
SOC. COMP. BOXES	IMPROVEMENT	1 950 (250*)	1 950 (250*)	1 950 (250*)
	NEW	2 730 (350*)	3 120 (400*)	3 510 (450*)
COMMERCIAL TOP PRICE		12 590 (100**)	15 108 (120**)	16 997 (135**)

Exchange rate: 949.5 Colombian Pesos/US Dollar. * UPAC—Constant-Acquisitive-Power Unit≈7,8 US Dollars. ** MMS—Minimum Monthly Salary≈126 US Dollars.

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MATERIAL MATTERS IN AFFORDABLE HOUSING

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ABSTRACT. Many developing countries are facing serious housing shortages and many alternatives are being proposed as potential solutions. The major stumbling block to the provision of housing is the (non) availability of financial support for the ill-housed and often unemployed. However, the way in which available money is used will influence the extent to which shelter can be provided successfully. This paper reviews the need for housing in Sub-Saharan Africa and considers the options that are open in terms of construction techniques and material availability. It discusses some of the more innovative and labour intensive approaches that merit closer examination with reference to their durability, thermal efficiency and social acceptability. The versatility and relatively low cost of cement based materials coupled with the potential for labour intensive construction techniques and the environmental conditions in the sub-continent makes them potentially important components of any concerted plan to alleviate the lack of affordable housing.

Keywords: Low cost housing, shelter, thermal performance, durability, appropriate technology, Southern Africa, fly ash, PFA, cement, OPC.

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INTRODUCTION

In the building of his shelter primitive man faces one supreme and absolute limitation: the impact of the environment in which he finds himself must be met by the materials which that environment affords. The environment is scarcely ever genial, and the building materials are often appallingly meagre in quantity or restricted in kind.(Fitch and Branch, 1960)

Emerging countries face similar problems with regard to materials of construction.

It has been suggested that as much as eighty percent of the world is underdeveloped or developing and it is said that the growth in the population of the world between now and 2050 could be as much as twice the total population (4, 5×10^9) of the world in 1980 *(Preston and Rees, 1994)*. It is certain that the demand for shelter is going to be very difficult to meet. The backlog of housing in South Africa has been variously estimated as more than one million units and it is likely to grow before the implementation of the national housing programme begins to take effect. In India the housing shortage in 1991 was estimated at 31 million units *(National Housing Policy, Government of India, 1992)*. Similar scenarios can be cited for the countries of sub-Saharan Africa and South and Central America.

The provision of housing is governed by the need for shelter, the availability of materials, the appropriateness (economic and physical) of the materials to the environmental demands and by the social and other customs of the housed.

Since the greatest demand for shelter is from the under-employed and the unemployed, the cost of the housing and the potential for job creation are of paramount importance. It has been argued that the creation of employment should take precedence since people with an income can and will make their own provision for shelter *(Morris, 1994)*.

In South Africa the greatest stumbling blocks to the provision of acceptable shelter have been legal (Groups Areas Act), lack of proclaimed and serviced land and the unavailability of appropriate finance. Nevertheless the technology employed does play a major role not only because different techniques have different immediate cost implications but also because they involve the community differently.

What is sought after, therefore, are building systems that are relatively cheap but durable and thermally acceptable as well as allowing participation by the community and thereby improving the cash economy of that community.

The South African experience with the provision of housing on a massive scale in the decade from 1950 to 1960 was only partially successful. It provided shelter, at low cost and of very basic standard, in what were planned and executed as temporary dormitories in accordance with the political ethos of the time. The ‘townships’ were not intended to be viable as towns or suburbs and lacked all community amenities. A repetition of this experience could not and should not be tolerated. However, the need is such that it may be necessary to build ‘mass’-housing in certain areas, using large contracting companies and even industrialised construction systems. Neither of these would meet the criteria of involving the community in the construction process and of keeping the money involved within the community.

Much is made of the potential for job creation of massive housing programmes but it might be argued that the reverse is also true, namely, that if unemployment can be reduced it will lead to the provision of housing for themselves by the newly employed.

(Morris, 1994) To give the unemployed housing that they cannot afford is to trap them in sheltered starvation.

The official approach to alleviating the South African housing shortage has undergone dramatic changes since the change in government in 1994. Initially reformist fervour led to unrealisable promises of relatively large, expensive houses for all. Gradually reality imposed itself on the authorities and the current policy is that all available means must be explored to provide shelter, whether this entails mass production of ‘formal’, though small, houses; small projects undertaken by ‘emerging’ builders with or without assistance in the management of the project or ‘self-build’ housing on sites that are provided with minimal services.

Much attention has been paid to creating a financial climate within which large or small loans can be made available to those who have a minimum of collateral. At the lowest end of the economic scale, outright grants are being made to provide the unemployed or indigent with a site that has been serviced (electricity, water and waste disposal) and on which he /she can erect a shelter that may be rebuilt or upgraded as the occupant’s financial position improves *(Cleobury J, 1995)*.

CONSTRUCTION TECHNOLOGY

During the era of mass-housing construction in 1950–1960, most of the houses were constructed of clay brick or concrete block walls on concrete strip foundations with galvanised steel or asbestos cement roofing *(NBRI, 1987)*. There were no floors and no internal doors, nor was there reticulated water, water borne sewerage or electricity provided.

This approach would not be acceptable now but the way in which the houses were constructed might. Although the labour content was high it was organised to ‘industrialise’ the process. Teams of unskilled workers were trained to carry out specific tasks, for example, one team would dig the foundations, another would cast the foundations, and a third would build up the corners. In this way the assembly line workers moved along the assembly line of houses.

Various attempts have been made to construct mass-housing in South Africa using industrialised systems of heavy-weight as well as with light-weight concrete panels without notable success. It is not clear whether the scale on which these projects were based was too small to justify the capital invested or whether the difficulties of maintaining dimensional accuracy were beyond the capability of the protagonists but it is likely that in the new circumstances there will be new attempts to use industrialised systems to meet the demand for housing.

What is certain is that because of the lack of an industry and construction tradition based on indigenous timber, housing construction will rely heavily on cement and concrete in one form or another. This applies to the whole of Southern Africa. Despite the very extensive plantations of exotic timber, South Africa cannot meet its own need for construction timber and the indigenous timber is both unsuitable and scarce. Malawi is facing such massive depletion of its timber resources resulting from the insatiable demand for fuel that it might have to follow India’s lead and ban the use of timber in the construction of governmental buildings. Zambia and Zimbabwe are in much the same position as South Africa whereas much of Namibia and Botswana is semi-desert or desert.

With the exception of the immediate vicinity of the coast, most of southern Africa can be described as a relatively high plateau (more than 1000 m above mean sea level) and typically continental conditions prevail with large diurnal temperature fluctuations. In the dry, cloudless winters the daily fluctuation often exceeds 20°C between minimum and maximum air temperatures. This in turn demands that the housing should be of high thermal mass so that a thermal fly-wheel effect can help to even out the disparities between interior night and day temperatures. It also requires that consideration be given to the orientation of the houses, the location of windows and the insulation of the roof structure. Research by the National Building Research Institute (NBRI) from 1960 to 1980 provided much valuable information on the optimization of the thermal performance of buildings under Southern African conditions *(Van Straaten J F, 1967; Higgs FS 1986)*. A recent Masters thesis submitted to the Chinese University of Hong Kong, uses computer simulation to investigate optimum construction techniques for a part of India that is hot and dry and therefore similar to Southern Africa. The housing design differs from what is customary in Africa but the need for mass is very evident in the author's conclusions. *(Shrinath T R, 1995).*

An added complication is that many deaths occur during winter due to the use of open braziers that are brought into informal and formal housing at night. In an attempt to conserve heat, ventilation is reduced to a minimum and people die of carbon monoxide poisoning.

Besides the health and comfort aspects of correct thermal design and construction there is the important aspect of the cost of heating such houses during the cold winter nights. The poor cannot afford to spend money on heating.

There is also the subjective yet important perception that whatever housing is created, it should not differ greatly from the more expensive housing people are used to seeing. This has limited the scope for innovation, and panel-type industrialised construction is not regarded highly since it does not appear to have the solidity of a brick-and-mortar house.

It is clear therefore that many factors conspire to make cement and concrete important in the provision of housing for the many homeless people in Africa.

CEMENT AND CONCRETE IN HOUSE CONSTRUCTION

As has already been mentioned formal house construction in Southern Africa is traditionally based on concrete strip foundations and clay brick or concrete block and mortar walls. Roofs are either pitched timber trusses or mono-pitch timber beams covered with galvanised steel or fibre-cement sheets. Window and door frames are of steel and doors are either solid or hollow core timber.

Increasingly new residential areas will be laid out on unsuitable or unstable soils simply because much of the better founding has already been used. Much of Southern Africa is overlain by layers of desiccated clays which swell when they become moist as they inevitably do when covered by buildings. The NBRI measured swelling of as much as 200 mm in areas of the Free State province of South Africa *(NBRI, 1985)*. Other areas overlie extensive dolomitic deposits and the formation of sink-holes is not uncommon. A complete reduction plant at a gold mine in the Western Transvaal disappeared into such a sink hole some forty years ago.

The consequence is that much attention is being paid to the foundations needed to overcome the problems caused by, for example, heaving clay. A variety of raft-like foundation designs have been developed and so-called waffle-rafts can be produced at costs that compare well with the more traditional strip foundations *(Williams A A B and Donaldson G W, 1980, Harrison N C, 1994)*. They are purpose designed in reinforced in-situ concrete and ensure that the superstructure will not crack disastrously.

Innovative approaches to the construction of walls include the use of in-situ cast walls of sand/cement ‘concrete’. The NBRI developed this system under the name of ‘BRISC’ (Building Research In-Situ Concrete) for use at Kabokweni in the eastern Transvaal and showed that acceptable accommodation could be provided at an affordable price *(Scott T W, 1983)*. The design used plywood form work designed to be manhandled and requiring no cranage or other mechanical manipulation.

The relatively dry mortar was then tamped into the forms to produce monolithic walls with window and doorframes pre-positioned and built in.

This design had an interesting sequel when the NBRI was called on to help the government of the Comoros to develop a building system that would avoid the destruction of the coral and the beaches that constitute a major source of tourist income. The custom had been for coral to be calcined to produce quicklime and combined with beach sand to produce an indigenous hydraulic cement. The islands are volcanic and there is little other building material available. The solution proposed was that cement should be imported and combined with volcanic gravel to form a coarse mortar. Formwork, similar to that used in the BRISC system but even easier to handle, was designed. Large lumps of volcanic rock were placed in the forms by hand and the coarse mortar was tamped around these ‘plums’ to form the walls. Once the formwork had been stripped, a thin rendering of cement mortar provided a surface finish that was acceptably smooth and easy to paint.

Evaluation of this technique showed that it used only half as much cement as would have been used had the same house been built of concrete blocks and mortar *(NBRI, 1986)*.

This development was a good example of using the materials available, in this case volcanic gravel and broken rock, (avoiding the need for materials that are scarce, and reducing the need for expensive imported products) and allowing the use of local labour instead of sophisticated equipment. Following this theme and remembering the statement by Fitch and Branch that the need for shelter must be met by the materials that the environment affords, one needs to examine the use of soil as a building material.

Soil has been used as a building material throughout the history of humankind and it is still being used as adobe or pisé-de-terre (rammed earth) and there are examples of such use in many parts of Africa. One of the disadvantages is that sun-dried soil blocks or mortar are not resistant to water. For many in Africa it is standard practice to repair their homes or other structures annually to counteract the damage caused by rain. In Malawi the thatched roofs that are typical of traditional home building, are extended to almost a metre beyond the perimeter walls to protect them from direct rain induced erosion.

If, however, the soil-based building elements, whether they be of rammed earth, sun dried blocks or soil-and-cowdung mortars, could be stabilised against the effects of water, the criteria set by Fitch and Branch may be met.

Much work has already been done on the production of soil-based building materials and the Building Research Establishment (BRE) in Britain and various researchers in Australia, among many others, have published results of their investigations. BRE developed a manually operated block-making machine for stabilised-soil blocks (BREPAK) *(BRE, 1987)* and Middleton *(1975)* in Australia developed a device for testing the resistance of soil based building materials to standard conditions of water spray. Internationally several manufacturers have produced more or less sophisticated machines for producing soil or soil-cement masonry units and South Africa is no exception. There are at least two manufacturers of machines for producing building blocks currently in the market.

In the laboratories of the University of the Witwatersrand the production of adequately strong and water resistant walls of stabilised earth in the form of rammed earth and of hydraulically compressed blocks has been investigated. The soils were characterised by the usual techniques of soils analysis and the range of cement concentrations that would ensure durability determined. The results indicate that at relatively low levels of cement content (between 6 and 8% of an 85% OPC/15% PFA blend known as PC15PFA), a wide range of soils could be satisfactorily stabilised. *(Roberg C W, 1993; Blight G F, 1994)*. The potential therefore exists for cement-stabilised soil to play a major role in the alleviation of the housing shortage in Africa since the equipment is relatively inexpensive and simple to operate, the production of masonry units or rammed earth walls labour intensive and the basic materials readily available.

A development that has recently come to our attention is a simple hand-operated machine for the production of concrete roof tiles. The use of concrete roof tiles manufactured in sophisticated factories is well established and provides attractive and durable roofing but the advent of a simple device that allows rural development of small scale manufacturing facilities could be valuable in its creation of employment and entrepreneurship. The device has been used extensively in Malawi and Zimbabwe and the quality of the product appears to meet the needs of the users. Nevertheless, the South African manufacturer of the tile making machine has submitted tiles to the South African Bureau of Standards (SABS) for evaluation to ensure that the final product will resist the often destructive hailstorms that strike the inland areas of Southern Africa.

There are many other innovative techniques that are being actively promoted and most of them rely on the use of OPC or OPC/PFA blends. Because South Africa derives about 90% of its electrical energy from coal-fired thermal power stations and the ash content of the coal is high (>30%), the amount of PFA produced annual is measured in tens of millions of tonnes. The incorporation of PFA in cement blends has become almost universal. In addition the use of coarser grades of ash as fillers or fine aggregate is becoming widespread.

An example of this in the housing field is provided by the commercially developed version of ‘wattle-and-daub’ construction that uses expanded steel mesh to replace the lathes of wattle and a cement and PFA mortar to create the wall. The cementitious material is applied to the metal on site by hand or it may be sprayed on. The wall panels thus formed are nominally 4 metres by 2,4 metres in size and the claim is made that a house of modest size can be constructed in one day. The structure is founded on steel rods driven into the soil and attached to horizontal steel U-channels that support the walls. The PFA is necessary as it aids pumpability for 'Gunniting' and reduces that cost of the cementitious mortar.

It has been assumed that concrete block or ash block construction is the cheapest way to erect low cost single storey homes and much has been made of the labour intensive local production of such blocks by the prospective home owner/builders as a means of empowering the homeless to contribute to their housing and to do it in such a way that the money involved is circulated within the local community. By using suitable, screened clinker, sand, coarse PFA and classified fly ash with cement as the binder it is possible to produce blocks of adequate strength and durability that are 95% ash-based. It is claimed by the protagonists of this 'wattle-and-daub' system that it can produce walls at a price that is only two thirds the cost per square metre of concrete block walls.

Other approaches to the provision of housing include renewed interest in foamed concrete. The Department of civil Engineering at the University of Pretoria recently reported the construction of a $24m^2$ house on the basis of casting 6m×4m foamed concrete (800 to 1400 $kg.m^{3)}$, including doors and windows where required, and tilting the cured slabs into position. The concept of tilt-up construction is not new but we believe the use of foamed concrete in this form of construction may well be (The Civil Engineering Contractor, 1995).

CONCLUSIONS

The need for housing throughout the developing world is great and there is no simple solution to providing such housing. The environmental requirements differ according to location and climate, there is no single construction technique that will meet all needs and the appropriate solutions will be as varied as the circumstances. Nevertheless it is clear that cement and cementitious materials including pulverised fuel ash will play a major role in addressing the shortage of housing.

In South Africa many different approaches will be adopted and used in parallel and there is no reason to believe that it will be different in other developing countries but the participation in the provision of housing by the those who need the housing will be crucial to the success of any endeavour.

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CEMENT STABILISATION OF SOIL FOR THE PRODUCTION OF BUILDING BLOCKS

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South Africa

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ABSTRACT. The use of soil as a building material is as old as humanity itself. Even now it is used in various parts of the world in the form of sun-dried blocks or bricks, as adobe and as pisé-de-terre (rammed earth).

The urgent shortage of housing in Southern Africa has re-awakened interest in soil as a building material as it offers the advantages of ready availability, labour intensive construction techniques and the potential for keeping much of the money in the community.

This paper considers the historical use of soil and discusses some of the research that has been carried out in the Department of Building and Quantity Surveying of the University of the Witwatersrand. It also discusses the thermal advantages, the durability problems and some solutions before suggesting some approaches that appear promising.

Keywords: Soil, stabilisation, low cost housing, ordinary portland cement (OPC), pulverised fuel ash (PFA), construction, appropriate technology.

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INTRODUCTION

It has been suggested that as much as 80% of the world is underdeveloped or developing and it is said that the growth in the population of the world between now and 2050 could be as much as twice the total population ($4,5\times10^9$) in 1980 *(Preston and Rees, 1994)*. It is certain that the demand for shelter is going to be very difficult to meet. The backlog of housing in South Africa has been variously estimated as more than one million units and it is likely to grow before the implementation of the national housing programme begins to take effect. In India the housing shortage in 1991 was estimated at 31 million units *(National Housing Policy, Government of India, 1992)*. Similar scenarios can be cited for the countries of sub-Saharan Africa and South and Central America.

Since the greatest demand for shelter is from the under-employed and the unemployed, the cost of the housing and the potential for job creation are of paramount importance. It has been argued that the creation of employment should take precedence since people with an income can and will make their own provision for shelter *(Morris, 1994)*.

In South Africa the greatest stumbling blocks to the provision of acceptable shelter have been legal/political (Group Areas Act), lack of proclaimed and serviced land and the unavailability of appropriate finance. Nevertheless the technology employed does play a major role not only because different techniques have different immediate cost implications but also because they involve the community differently. In addition serious concerns have been expressed about the ability of the local materials industry to meet the expected demand for products.

What are sought after, therefore, are building systems that are relatively cheap but durable and thermally acceptable as well as allowing participation by the community and thereby improving the cash economy of that community.

SOIL AS BUILDING MATERIAL

The use of soil as a building material is as old as humanity itself and sun-dried bricks were used in Ancient Egypt and Mesopotamia. It was used in the south western United States by the cliff-dwellers before the arrival of Europeans. In Malawi the use of sun-dried clay building blocks is fairly common and the traditional practice of extending the thatched roof on all sides to provide an eave-overhang of about a metre is a pragmatic solution to the problem of rain-induced erosion of the clay-block walls. It incidentally also improves the thermal performance of the building as it shades the walls from the tropical sun.

In southern Africa sun-dried clay bricks or cement stabilised pisé-de-terre (rammed earth) appear to have been stigmatised as unacceptably primitive materials whereas in parts of the south western United States (Arizona and New Mexico) adobe houses are sought after, mainly because of the excellent thermal comfort adobe construction provides. However, research into the use of clay as a building material has been widespread and there are a number of initiatives aimed at re-establishing the use of clay as a building material *(McHenry, 1984, United Nations Centre for Human Settlements, 1984, Roberg, 1993)*.

The problem of water resistance is likely to restrict the use of unfired clay building materials seriously and it provided the motivation for the research that was undertaken by the Department of Building and Quantity Surveying of the University of the Witwatersrand.

Cement-stabilised soil masonry units

The concept of stabilisation of soil by the addition of cement (Ordinary Portland Cement (OPC) or blends of OPC and Pulverised Fuel Ash (PFA)) is readily accepted for the construction of base courses for roads but has hiterto not been widely applied to the production of masonry units *(Ballantine and Rossouw, 1972)*.

Roberg *(1993)* investigated the use of stabilised soil in a rammed earth type construction technique. He selected a variety of soils from around Johannesburg and characterised them in terms of their physical properties, using the full range of soil science techniques. On the basis of the known properties he then determined the optimum moisture content to ensure maximum compaction and the optimum soil to hydraulic cement (ie 85% OPC/15% PFA known as PC15PFA) ratio for each of the soils. These mixtures were then manually tamped into wooden formwork to create walls.

His findings were that relatively low cement to soil ratios (6% to 8%) and water contents of between 12% and 14% produced rammed earth walls that were strong enough for single storey buildings and resistant to a severe water-erosion test *(Middleton, 1987)*.

There are at least two commercial concerns in South Africa that are marketing devices for the manufacture of building blocks composed of stabilised earth. The basic concept was well described by the Building Research Establishment (BRE) in the UK *(Building Research Establishment, 1987)*. The production of the blocks may be carried out manually using a simple device such as BRE's BREPAK or by motor-driven hydraulically operated moulding machines of varying degrees of sophistication. At present the degree of stabilisation (by the addition of cement or lime) is a matter of experience. It would make the systems more acceptable to a sceptical community and to those financing the housing if it could be demonstrated that the appropriate level of stabilisation for different soils falls within a relatively broad range and is therefore not critical to the performance of blocks made of stabilised soil.

Consequently Blight *(1994)* used tests similar to those used by Roberg to characterise two soils; the one was a quaternary sand with 14% clay and 3% gravel and the other a decomposed granite with only 4% clay and 40% gravel. To these soils he added PC15PFA in proportions ranging from zero to 8% after determining the optimum moisture content for each soil. The stabilised soils were then hydraulically compressed in a mould under a pressure of some 8 MPa using equipment supplied by Hydraform Concepts (Pty) Ltd. The product is a block that is profiled on four of the six faces so that it may be dry stacked since it interlocks with adjacent blocks.

The blocks were subjected to six different tests, namely

Unconfined compressive strength
Standard penetration
Linear shrinkage
Accelerated erosion

Wet/dry durability
Water absorptivity

Whenever possible unstabilised blocks (zero % cement) were included in the tests but since they disintegrated upon exposure to water they could not be tested for standard penetration, wet/dry durability or water absorptivity.

The blocks were allowed to 'cure' under plastic sheeting for three days, thereafter the unstabilised blocks were allowed to dry out, uncovered, while the stabilised blocks were kept under polyethylene sheeting until they were tested for unconfined compressive strength after four days.

Despite some variations in the rate and manner of strength development depending on the type of soil, Figure 1 and Figure 2 show that all the stabilised blocks, irrespective of the extent of stabilisation, reached acceptable strengths (≥3MPa) within seven days of manufacture. 'It may be concluded that the degree of stabilisation has a direct linear effect on the strength of blocks made of soils similar to those used in this study. Blocks which are stabilised by the addition of 4% PC15PFA will reach higher 28 days strengths than those which are unstabilised' *(Blight, 1994)*.

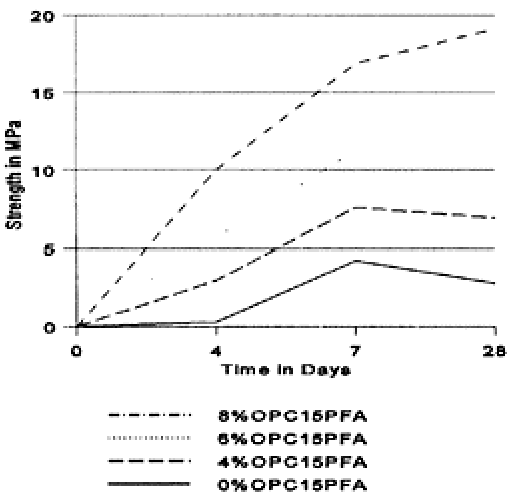

Fig 1 Quaternary Sand

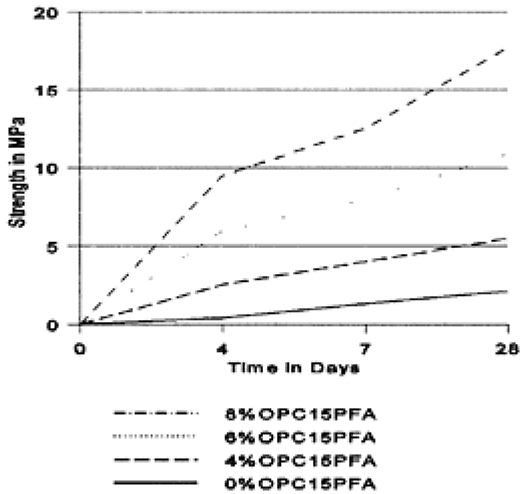

Fig 2 Decomposed Granite

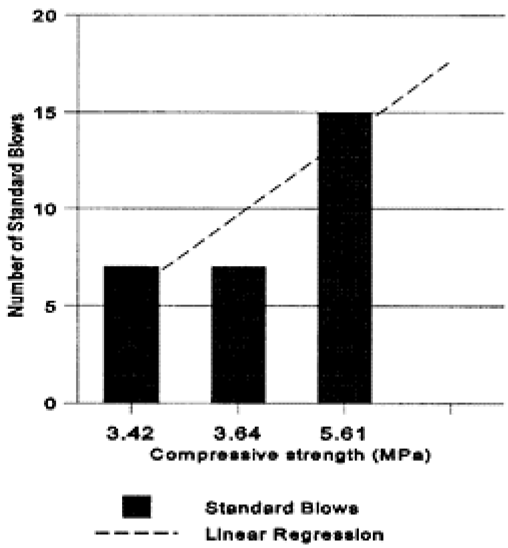

Fig 3 Quatenary Sand

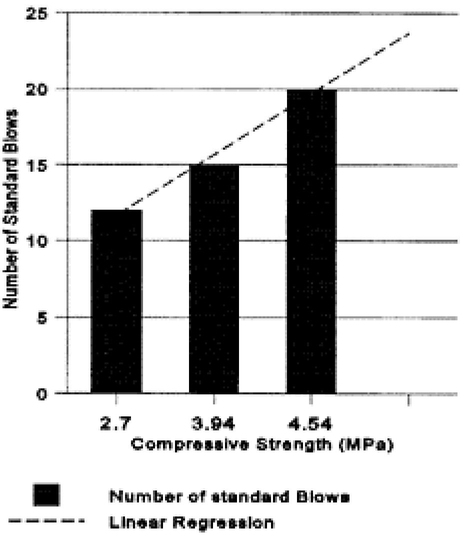

Fig 4 Decomposed Granite

The standard penetration test is a site-applicable test described by Hydraform *(1994)* as a form of quality control. It consists of the correlation with the unconfined compressive strength of the block, of the number of blows from a standard hammer required to drive a standard spike into the block to a pre-determined depth. After seven days of curing under plastic the block is soaked in water for eighteen hours. The block is placed on a suitably plane and hard surface and the spike is driven into the block at predetermined positions. The average number of standard blows required to achieve the desired penetration is then calculated and compared to the unconfined compressive strength of blocks of the same composition and the same age.

The results were interesting as they indicate a correlation between compressive strength and penetration but the correlation differs with the type of soil. Figure 3 shows how the results for the quaternary sand and the decomposed granite diverge with increasing strength of the blocks. The penetration test can therefore only be used to establish uniformity within the blocks produced using one specific soil type.

The linear shrinkage tests were intended to examine the rate of shrinkage and its dependence on the degree of stabilisation as well as on the age of the blocks. Immediately after manufacture aluminium targets were affixed to the blocks and their positions were monitored using a Demec gauge over the period of the test. Figure 5 and Figure 6 show how the shrinkage on the unstabilised blocks, whether quaternary sand or decomposed granite, was greater than for the stabilised blocks and that the degree of stabilisation made little difference to the shrinkage for both soils. The quaternary sand does shrink more than the decomposed granite presumably because of the larger grain size in the decomposed granite. The maximum shrinkage in the stabilised blocks represents only 2, 8 mm in a block that is nominally 250 mm long.

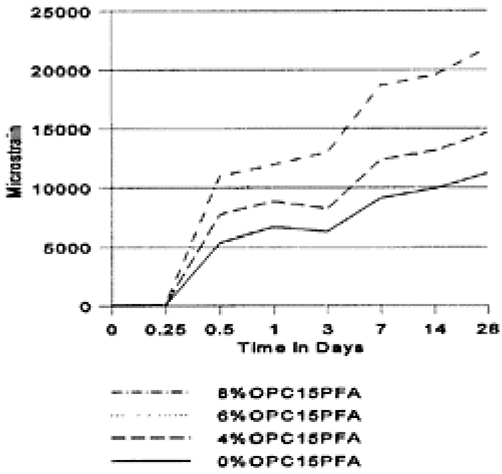

Fig 5 Shrinkage of Q Sand

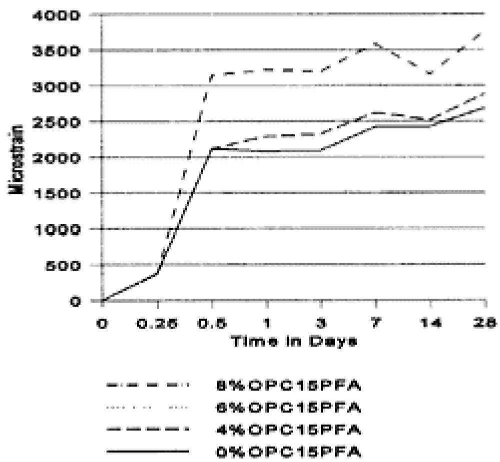

Fig 6 Shrinkage Dec.Granite

The accelerated water erosion test used a device designed by the Building Research Division of the CSIRO (Australia) and described by Middleton *(1987).* In this test a spray of water (at a prescribed volume per minute and prescribed pressure) through a standard nozzle is directed at the test object at a prescribed distance for one hour. The test is interrupted briefly every fifteen minutes to examine the test object for erosion.

The results indicate that unstabilised blocks of either soil type are unsuitable for use where they might be exposed to rain whereas the addition of as little as 4% of PC15PFA improves the performance of the blocks remarkably. Such deterioration as occurred was limited to spot erosion that did not develop further and 'It would seem that unless a high standard of finish was demanded, the addition of 4% PC15PFA by dry mass to most soils, will provide a sufficiently durable block' *(Blight, 1994).*

The wet/dry durability test was based on a test for engineering materials *(TMH1, 1979).* The blocks were cured for seven days and then subjected to twelve cycles of wire brushing, wetting for five hours and drying for 42 hours before repeating the cycle. By recording the mass and moisture content of the block before starting the test and again after the twelfth cycle the amount of material lost can be determined.

Not unexpectedly there is an inverse relationship between the amount of material lost and the degree of stabilisation and as with some of the other tests there was a difference between the performance of the two different soils. Since the test is difficult to relate to the demands likely to be placed on masonry units in home building, only the generalised conclusion that higher degrees of stabilisation will provide greater abrasion resistance and that coarser soils tend to be more abrasion resistant can be drawn from this test.

The water absorptivity tests were not very successful for various reasons but it was evident that the coarser material (decomposed granite) absorbed more water than the finer material and that after two hours the rate of water absorption dropped sharply in both cases reaching values of between 2% and 4,5%. Increasing the degree of stabilisation also decreased the amount of water absorbed.

The overall conclusions of this set of investigations were that the addition of as little as 4% PC15PFA to most soils would allow the production of hydraulically compressed masonry elements of adequate strength and durability for the construction of single storey domestic dwellings.

Thermal performance

Since the great majority of the housed and unhoused South Africans live on the high interior plateau (≥1000 metres above mean sea level) known as the Highveld where diurnal ambient temperature fluctuations often exceed 20°C the thermal mass of the construction materials becomes important. Light weight materials have little thermal inertia and the temperatures of houses built of them are likely not only to follow the variations in ambient temperature but to exceed them. In contrast heavy weight materials such as brick-and-mortar or soil-cement blocks have a considerable capacity for absorbing heat during the day and releasing it at night thereby acting as a thermal flywheel and reducing the fluctations in the internal temperatures of the houses *(Van Straaten and Lotz, 1972, Wentzel, 1981).*

Since low income and no-income families can in any case ill-afford the cost of heating their homes in winter, it is of considerable importance that the hundreds of thousands of houses that are to be built under the South African Government's new housing initiative be as energy efficient as possible.

The properties of soil/cement masonry units, manufactured on site are such that they should be seriously considered as one of the ways to meet the housing needs of developing countries. A system such as that examined in our laboratories might offer an approach that, because of its simplicity, would allow relatively unskilled workers to erect durable, thermally efficient homes at reasonable cost, thereby developing their skills and contributing mterially to relieving the housing shortage in the developing world.

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AN INNOVATIVE LABOUR-INTENSIVE METHOD FOR CONSTRUCTION OF ARCH BRIDGES USING UNCUT STONE AND MORTAR

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Appropriate Concrete Technology. Edited by R K Dhir and M J McCarthy. Published in 1996 by E & FN Spon, 2–6 Boundary Row, London SE1 8HN, UK. ISBN 0 419 21470 4.

ABSTRACT. Lack of adequate infrastructure in rural South African communities, coupled with the abundance of uneducated and unemployed people, has prompted authorities to encourage labour-intensive construction methods. One such technique, using natural uncut stone and cement mortar, is described. Inadequate knowledge of the material strength and magnitude of stresses experienced in these structures has prompted investigation aimed at formulating guidelines for the design and construction of more efficient structures of known reliability. A procedure has been developed for testing the compressive strength of large stone and mortar cubes. Some of the failure characteristics indicate that incorporation of minor changes to current practice may well yield a material with superior properties. Measured live-load-induced stresses are used to validate a theoretical finite element model, to evaluate alternative arch shapes, providing greater spans and reducing the volume of material required.

Keywords: Labour-intensive, Uncut stone masonry, Cement mortar, Arch bridge, Cube strength, Finite element analysis.

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INTRODUCTION

Many impoverished communities throughout Southern Africa experience periods of isolation when floods wash away parts of their road infrastructure, or just make road use impossible during the rainy seasons[1].

In 1992, the former Lebowa Government's Roads and Bridges Department called for tenders for the installation of bridges as part of its road management system. The engineers were prompted, because of a materials supply problem, to explore alternative construction methods. A technique, similar to that used in Zimbabwe[2,3] to build stone-arch causeways, was proposed and successfully implemented. In addition to overcoming the immediate problem of a materials shortage, the new construction method also provided several social benefits:

a) This construction method is highly labour-intensive, providing jobs for hundreds of local people who would otherwise be destitute.
b) Most of the money spent on each bridge, is invested into the local community through wages, hiring of local subcontractors and local purchasing of material.
c) This building technique minimises energy input by using abundant local material and is therefore likely to be sustainable, thus empowering developing communities to provide their own transportation infrastructure.
d) The design philosophy of a rigid mass structure, aims to achieve very low stress levels, thereby increasing the tolerance of the structure to geometrical and material variations, so as to accommodate the relatively low level of local skills.

Many of these principles have subsequently been recognised, encouraged and implemented by the South African Government in its Reconstruction and Development Program[4] and the National Public Works Program[5].

DESCRIPTION OF CONSTRUCTION METHOD

Once the foundations are in place, sheets of corrugated iron are pre-cranked to the shape of the intrados and positioned over the foundations to provide temporary support to the arch span. In the case of large spans, these sheets are propped with gum poles to help bear the weight of the masonry until it becomes self-supporting (Fig 1). In some areas alternative centering using 50mm saplings, covered by old cement packets, and supported by temporary mud brick walls has been successfully deployed[1].

Figure 1 Construction of a typical bridge showing the temporary corrugated iron and gum pole centering used to form a 2, 7m radius arch.

Workers gather stones (typically weighing anywhere from 10–40kg), from the fields nearby. Sand from the river bed nearby is used to make a cement mortar. The cement: sand ratio (by volume) varies from 1:4 to 1:3. The manual assembly begins by embedding individual stones into a thick layer of mortar. The stones are not cut or dressed in any way, and are placed dry in intimate contact with one another. This procedure is followed; horizontal layer upon horizontal layer, until a solid structure results. As a precaution, a grid of high tensile reinforcing steel has been placed horizontally in both directions, immediately above the crown of the arch in the initial structures. This steel is placed in a thick bed of cement mortar, of a 1:3 cement: sand mix. Another bed of mortar (or preferably concrete) is used to even out the bridge deck to form a trafficable surface.

STRENGTH OF STONE-MORTAR MATERIAL

Cube Tests

As traditional 100mm and 150mm concrete testing cubes are too small to accommodate even the smaller stones used in this material, an alternative test was developed by Rankine[(6)]. For ease of fabrication, the traditional cube shape was maintained, but the overall dimensions were increased to accommodate the large stones. Initially cubes of 300mm, 400mm and 500mm were chosen as a means of exploring different possible sizes. The 300mm size proved too small to accommodate all but the very smallest stones and the 400mm size only permitted one, or at most two, of the larger stones, Thus, it was decided that the 500mm size, shown in Fig 2, was the most representative of the material and has subsequently been adopted for further testing.

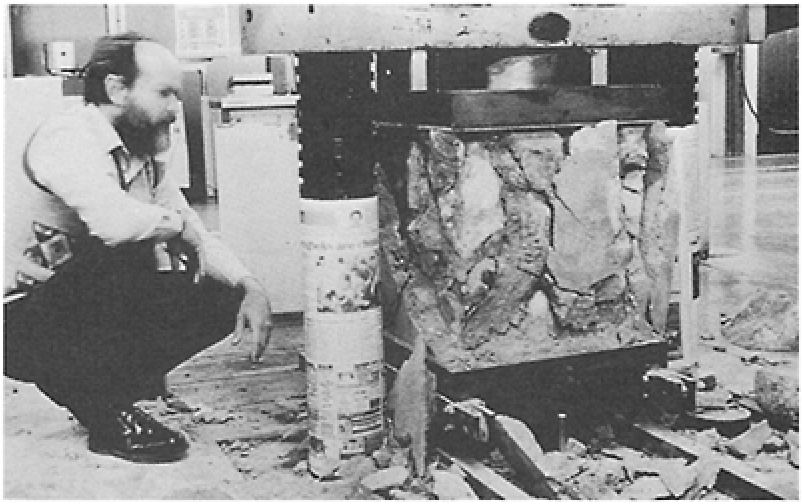

Figure 2 Large scale compression test on a 500mm stone-mortar cube weighing 300kg.

Cube stresses

Rankine et al[7] describe some large cube tests which were carried out as a preliminary indicator of the strength of the materials used for these bridges. The cubes were made on site, using weathered hand gathered stones and the same procedures as were used for the bridges themselves. A subsequent series of tests[8] using quarried dolomite rock (graded in various combinations using individual stones weighing between 1 40Kg), and the same strength mortar has produced a stronger material. The 28 day compressive strength of these cubes is presented in Table 1. The weathered stones with a friable chalky surface do not appear to provide as good a substrate for the bonding of cement mortar, as do quarried rocks with fresh faces.

Table 1 Results of compressive tests.

Cube size (mm)	Mass (Kg)	Failure load (KN)	Failure stress (MPa)
300	60	1260	14, 0
400	149	1435	9, 0
500	–	2406	9, 6

*500	–	3138	12, 6
*500	–	3293	13, 2

The first three specimens contained hand gathered stones with weathered faces. Specimens prefixed with an asterisk represent an average of three test cubes containing quarried dolomite (with fresh clean surfaces). The 28 day 100mm cube mortar strength for all specimens was between 14 and 15 MPa.

Consequences of orientation of stones

To simulate the compressive thrust generated above the crown of the arch, load was applied to the cube specimens perpendicular to the plane of casting. (As flat elongated stones are currently positioned horizontally, axial cornpressive stresses over the crown of the arch, tends to be applied parallel to the stones' longitudinal axes). Where failure of the specimens appeared to be initiated by bond failure at the stone-mortar interface, this often resulted in a wedging or cleaving action by the long slender stones which lay parallel to the axis of applied load. It is interesting to note that similar observations have been recorded in a parallel investigation(8) of physical model melanges during a study of rock mechanics.

Large flat stone surfaces, when placed horizontally, were observed to exacerbate the problem of early bond failure. The horizontal surfaces tended to trap bleed water on its upward migration and form a void between the mortar and the stone. Hence, it is proposed that further testing establish quantitatively the effect on cornpressive strength of placing elongated stones radially rather than horizontally.

STRESSES WITHIN THE ARCH

Measurements in-situ

To gain an initial understanding of the magnitude and nature of the stresses and strains within the arch structure, measuring devices were made by fixing electronic strain gauges onto concrete prisms as shown in Fig 3. These were then built into arches during construction. At a stage when the bridge became trafficable, a truck with an axle load of 100 KN, was driven across the bridge (see Fig 4), to record the stresses in each device at various axle positions. The first series of stress measurements confirmed that, the live-load-induced strains, and hence stresses were very low. The measured tensile and compressive stresses, induced by this live- load did not exceed 0,074 MPa. It was evident that very sensitive instrumentation was needed to provide more reliable data. Subsequently, a mechanical strain muliplying device has been developed(9) in an effort to amplify the the strain before it is recorded. This results in a more sensitive signal, without any sacrifice in clarity and without generating more noise. The data gathered will be used for validation of further theoretical models.

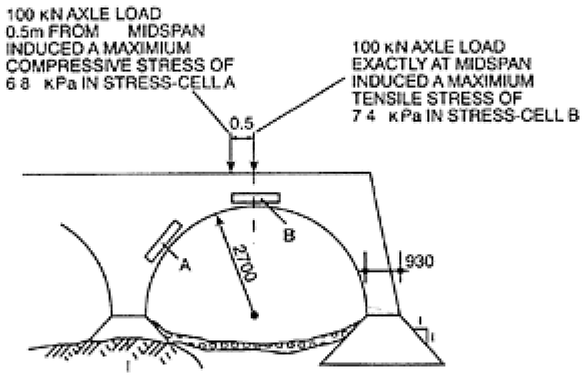

Figure 3 Recorded induced stresses in a 2, 7m radius arch with corresponding axle positions

Figure 4 Live-load-induced strain test

Finite element analysis

Because of the inherent difficulties in measuring self-weight-induced stresses, it has so far only been possible to predict these values theoretically. A preliminary finite element model has predicted self-weight-induced stresses of up to 0, 6 MPa, in the structure illustrated in Fig 1. Only once this model has been fully validated by confirmation of predicted live-load-induced stresses with measured live-load-induced stresses, can this value be safely relied upon. Thereafter, it is envisaged that the finite element model may be confidently used as a powerful exploratory tool for modelling new structures to be built from this material.

COMPARISON OF STRENGTH AND STRESSES

The low measured and predicted stresses within these structures, compared with the measured strength of the material, provides some temptation for speculation that an adequate margin of safety against compression failure exists. Preliminary indications suggest that allowable stresses should be limited to below one MPa, since traditionally, masonry arches have been assigned very high factors of safety against crushing of between 10 and 40[(10)]. It is also prudent to remember that the greatest contributor of stress in these structures appears to be the theoretically predicted self-weight-induced stress (which presently remains to be validated).

CONCLUSION

This labour-intensive method of bridge construction has fulfilled a dual function of providing infrastructure and creating job employment for hundreds of destitute people. A comprehensive set of design and construction guidelines are considered a prerequisite to structural engineers widely adopting this initiative. Preliminary research indicates that minor changes to current practice may result in a material with superior properties. Blasted quarried rocks, as opposed to weathered rocks, appear to produce a stronger material. Further research is proposed to quantify the effect of or ientation of stones with respect to the structures' stress trajectories and the use of finite element models as exploratory tools for predicting stresses in proposed structures is recommended.

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MANPOWER MOTIVATION IMPROVES QUALITY IN CONSTRUCTION

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ABSTRACT. Construction is a labour intensive process and manpower in one of the productive resources in it. Quality is directly related to the performance which is a function of motivational level of workers. Legitimate motivation to the manpower results in the improvement of quality in construction products. The study is based on the data collected from 24 different construction projects sponsored by the government. The data are analysed considering factors affecting morale and motivation of workers and their effect on quality of construction. The results of the survey report are presented in this paper. On the basis of data collected, the authors conclude that construction management, legitimate facilities including job satisfaction, proper training, and presence of some motivational plans etc., result in higher level of performance of workers which ultimately results in improvement of quality in construction. The authors also suggest suitable methods regarding motivation and co-ordination of manpower to achieve higher levels of quality in construction.

Keywords: Manpower, motivation techniques, construction, quality improvements

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INTRODUCTION

Quality is generally defined as “conformance to a standard of performance”[1]. One of the major factors in determining the human performance is their motivational level. The acceptable or normal level of performance can be raised to a considerably higher value if the workers are motivated[7]. Construction is a group activity that depends largely, not only upon the individual worker’s performance but on the coordinated effort of all the members of the Company. In order to improve the performance of workers, an appreciation of the functioning of the manpower and necessary motivation is required. Chishty and Choudhary concluded that Risk management ensures quality in construction projects[3], Chaudhary M.A. studied the manpower training and its effects upon efficiency of construction projects[4]. Effect of Time requirement upon Economy of construction projects has also been illustrated by M.A.Chaudhary and others[5]. In construction industry, quality is a major part of productivity. Poor quality work often results in rejection and rework which affects productivity and in turn results in time and cost overrun. Since labor costs comprise between 25 to 40 percent of the total project cost, reduced labor cost demand some source of increased productivity[6]. A firm’s productivity is further influenced by production factors other than labor, such as equipment, materials, methods of construction and management. These resources, if properly managed and scheduled can be successfully transformed into productive uses by the human element. The quality of human performance depends much upon human motivation. One of the findings of the workshop “Quality in the Constructed Project” sponsored by American Society of Civil Engineers, was that quality problems are created by the people who lack pride in their work[2]. The situation may be attributed to the work environment, lack of proper management, worker’s training, skill, and of course, motivational level.

Motivational effects upon construction has been analyzed in detail by Maloney and James[8,9], but so far little has been done to correlate the motivation for quality improvement in construction projects. Maximum research work in this direction is based upon motivation theories which are not well accepted in the modern approaches. The recent approach is to introduce motivation programs for the increase of manpower productivity in constructions.

The objectives of the paper are

1) To get a feedback from construction projects about the need for construction motivation.
2) To introduce factors which can influence manpower motivation.
3) To examine the role of management.

RESEARCH METHODOLOGY

The data for this study were collected at 24 different Government sponsored projects, ranging in cost from Pak. Rs. 100 millions to Rs. 750 millions approximately. The projects selected for survey work were of different types and of different geographical regions of the country. The range for stage of completion was different. Some were on going whereas maximum percentage of these projects was already completed. The data collected at each of the project consisted questionnaire survey and the interviews of the personnel at the site. The questionnaire was prepared after consultation with experienced personnel attached with the construction work. A team of the undergraduate students were trained for the survey work and the interview work was properly demonstrated at site. The questionnaire mainly related the manpower motivational tools, quality assurance techniques and time period.

The survey team faced great difficulties in the collection of data. The response was not encouraging, as at most of the sites the management hesitated to provide required information. The relative information at some of the sites, was obtained from the lower staff as proper management was not available whereas other sites were not even aware of such motivational plans. At certain organized project sites, this type of survey was very much appreciated and even some additional methods/approaches were conveyed which were ultimately incorporated in the questionnaire.

DISCUSSION UPON RESULTS

In the processing of the collected data, the major interest was to identify the factors influencing motivation of manpower and their productive ability. Quality of construction projects is related to motivation, performance and satisfaction of the manpower. Graphs regarding the factors influencing the quality and efficiency of construction project are shown in figures 1 through 7.

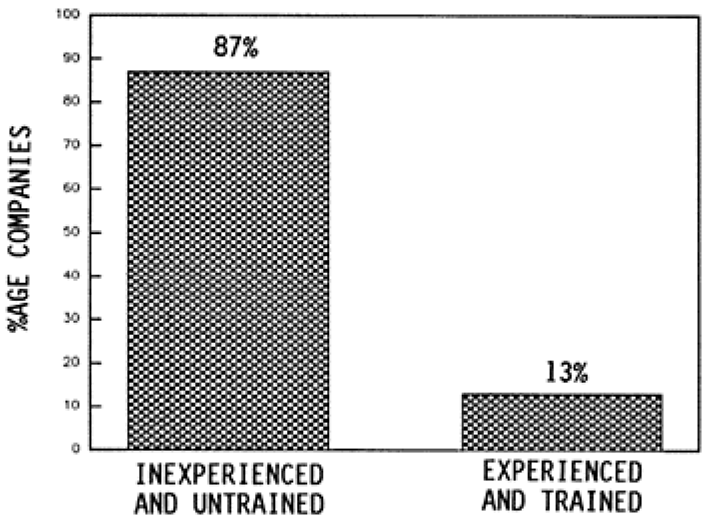

FIG. 1: TYPE OF MANAGER IN VARIOUS COMPANIES

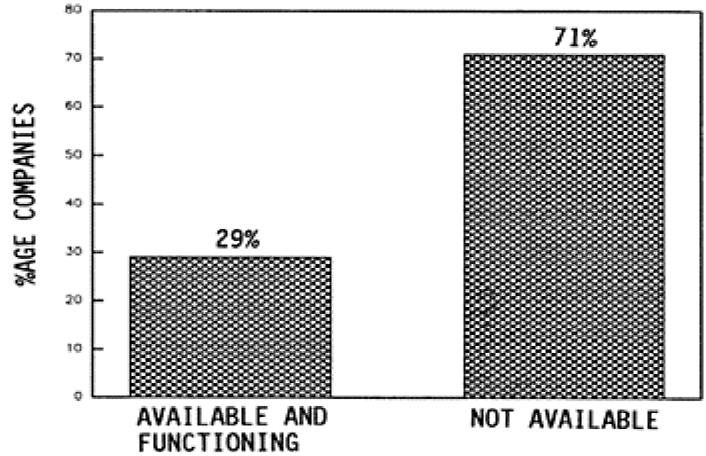

FIG. 2: REGULAR MANAGEMENT CELL IN VARIOUS COMPANIES

An inspection of the Figures 1 and 2 reveal that management practices are observed by a very small number of companies; only 13% of the companies have qualified, experienced and trained managers and 29% have regular management cell. Similarly, education and other training programs which impart a sense of confidence and result in higher level of performance and motivation in the workers, are also rare; only 21% of the companies surveyed provide educational training to their workers, see fig. 3.

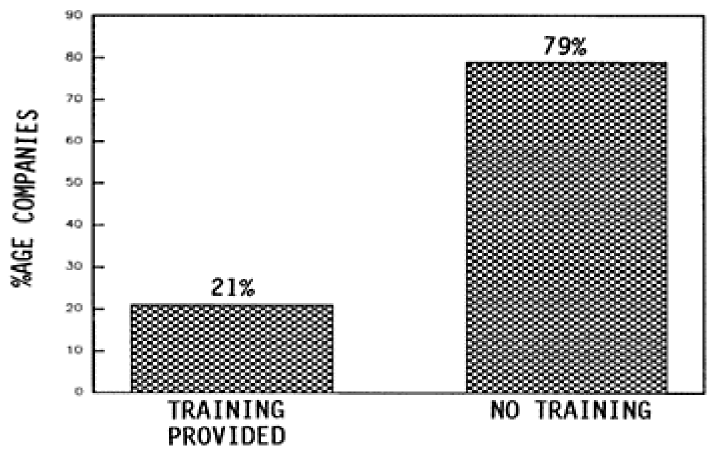

FIG. 3: TRAINING FACILITIES AT VARIOUS COMPANIES

Another very important factor that usually results in higher levels of performance is the safe working conditions and the presence of some incentive plan

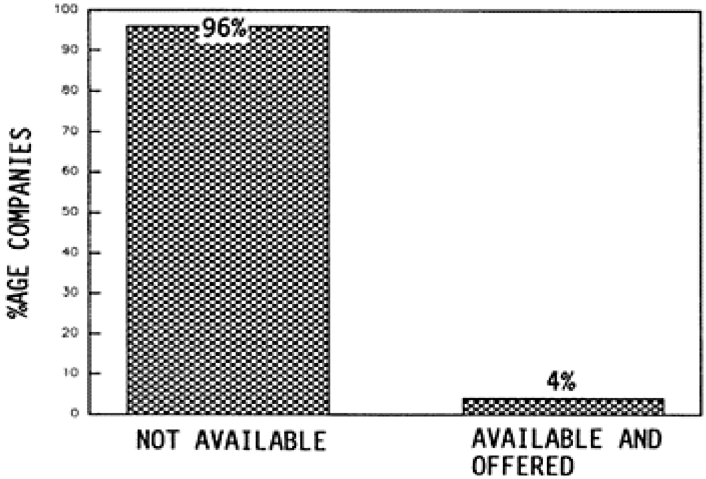

FIG. 4: MOTIVATIONAL PLANS WITH VARIOUS COMPANIES

further enhances it. However, the conditions regarding safety and motivational plans were not found proper and encouraging. From figures 4 and 5 it is clear that conditions of safety measures at construction sites are extremely poor; only one out of all twenty four companies surveyed has its own safety manual, and same company has motivational plans for its employees.

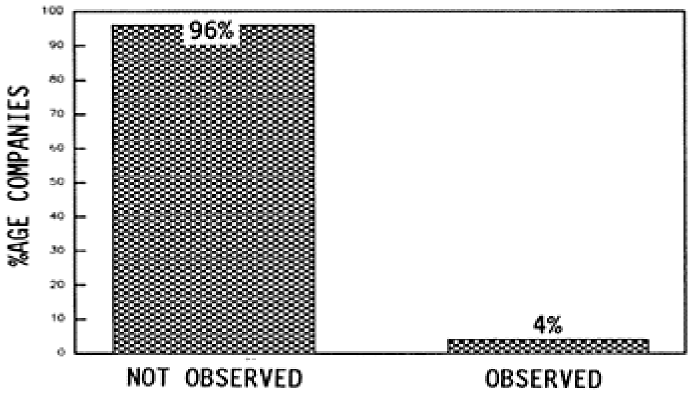

FIG. 5: SAFETY PRACTICES AT VARIOUS COMPANIES

Fig. 6 and Fig. 7 show the time and cost overrun for various projects. The projects which suffered maximum delay in completion and increase in cost are those which had a combination of the factors discussed above.

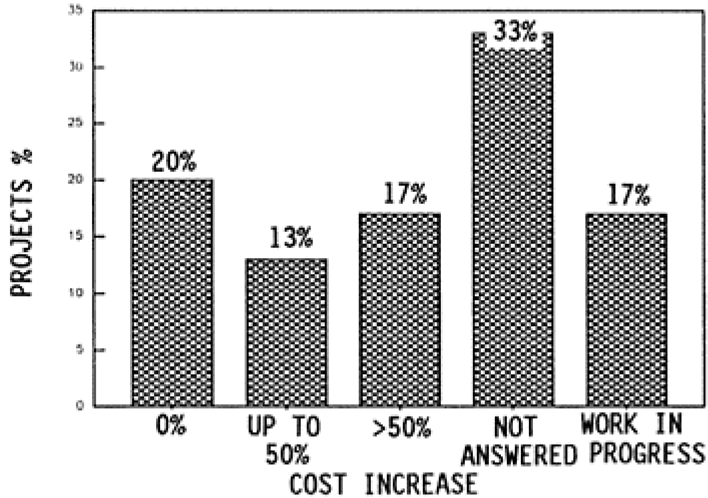

FIG. 6: COST OVERRUN FOR VARIOUS PROJECTS

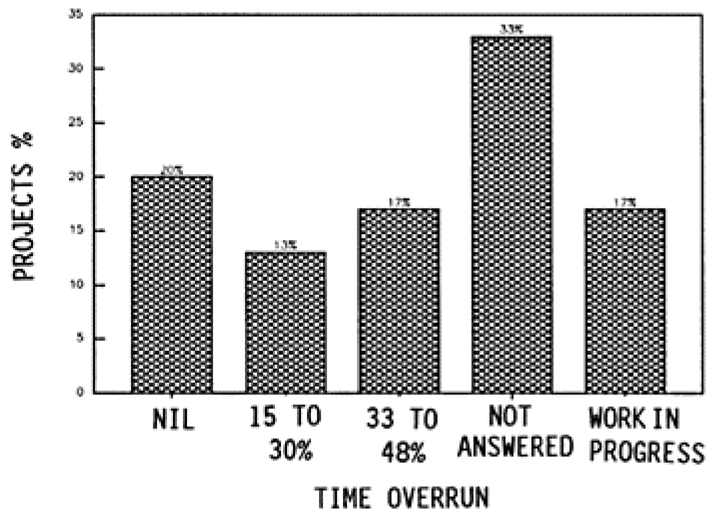

FIG. 7: DELAY IN COMPLETION TIME OF PROJECTS

Some of the other factors resulted in lower performance level include frequent Hiring and Firing, and Lack of Security against employment to workers. Similarly, on-site conditions like improper Site Layouts, Crew Interference, Unavailability of Required Materials at proper time affected severely the morale and motivation of workers. Frequent Design Changes and Untimely Quality Inspections resulted in poor workmanship.

On the other hand, on projects which were completed on time and within initial estimated costs, least amount of rework was reported. This was mainly due to the presence of Qualified Construction Manager on the site, and trained workers. Previous experience of workers on similar projects further enhanced the productivity and quality. On these projects, most of the equipment was owned by the owner and proper alternative arrangements in case of failure of an equipment were available. Similarly, safe working conditions and presence of incentives resulted in higher level of performance, productivity and hence improved quality.

CONCLUSIONS AND RECOMMENDATIONS

Based upon the collected data, the authors have reached at following conclusions and recommendations:

1. At most of the projects, the quality problems were mainly due to the factors affecting morale and motivation of the workers. Absence of qualified construction manager in the team was the most obvious one. To enhance the performance level of workers, inclusion of qualified project manager in the construction team must be a prerequisite. This may be included in the conditions of pre-qualification of the contractors, and minimum qualification of the manager must be specified at the time of award of the project.

2. Lack of education and training of workers resulted in poor quality work. In an industry like Construction, the problem may be solved, to a large extent, by issuing instructions about that day's work to Foremen and Supervisors every morning in 15–20 minutes before the start of the work. These instructions should include the scope of work for the day. These First-Line managers then may be advised to convey the same to the workers.

3. Lack or absence of incentives and motivational plans is another factor that affects quality/efficiency in construction. In this regard, it may prove to be quite encouraging that at the end of each milestone, those who have shown best performance or have improved during that time, be rewarded. These rewards may include financial or non-financial rewards or a combination of the two.

4. Unsafe working conditions lead to the feeling of fear because of the impending danger. This results in decreased concentration to work and hence low productivity and poor quality work. In order to enhance quality in construction, it is suggested that proper safety measures be ensured as per conditions of the project. The prevailing situation may be improved by educating the workers about safety everyday before the start of the work.

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APPRECIATING LOW-STRENGTH CONCRETE

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ABSTRACT. An overview of structural concrete in two countries in the Middle-East is presented. It is shown that in the two areas, one a corrosive environment, the correct selection of strength to be sought by designers will be in the lower range of values when the specific factors related to the locality are considered.

The results of field investigations of concrete condition and strength are given to demonstrate the reality of concrete actually produced and is supporting existing buildings. Protection from rain water was found to be more essential than strength to achieve the required durability. The way in which design can contribute to create more durable sections is discussed using a numerical example and a deduced correlation between UPV readings and concrete strength is used to show the minimum expected concrete strength.

The paper argues that the great attention paid to making concrete stronger in compression, though a necessity in a number of construction environments, is not needed in others. The main conclusion is that the role of lower strength concrete should still be appreciated, and related research needs to be continued.

Keywords: Compressive Strength, Durability, Corrosion, Chloride, Ultrasonic Pulse Velocity (UPV), Cube Sample Strength, Differential Settlement, Limestone, Contractor, Water Proofing, Design, Crack Width.

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research interest is focused on revising design and construction practices in order to overcome the prevailing durability problems in the Gulf Region.

INTRODUCTION

Strength -in addition to workability- has for many years been the main property of structural concrete looked after and pursued by those involved in the construction industry. The striving for an even stronger concrete has continued, mainly because of the need of the construction industry in the industrialized world in particular to increase the strength per weight ratio in the competition with other structural materials, namely, structural steel.

The cost of improving compressive strength of concrete is usually higher, the higher the strength and the extra cost has to be weighed against the benefits gained. The cost effective range of strength is usually determined from experience in a certain locality. Construction industries in some parts of the world are not well equipped to produce high strength concrete, but because of the international trend, engineers in these parts might occupy themselves with strength more than needed, concentrating less on other properties related to durability. This is particularly wrong in a rather severe environment like that in parts of the Middle-East where durability is turning into an obstacle that hinders the role of concrete as a structural material.

A number of investigations were carried out on concrete being the main structural material for ordinary houses and small buildings in two countries in the Middle-East, the findings of which will be presented in brief.

CONCRETE IN A NON-CORROSIVE ENVIRONMENT

Over a period of five years, 1985 to 1990, a comprehensive study was performed to find out the reasons for structural problems in a number of buildings, mainly residential four or five stories buildings and one or two stories school buildings in the suburbs of the city of Damascus, Syria. In the investigations, destructive and nondestructive methods were used to assess the quality of concrete. The major finding regarding strength was that the value in these buildings was between 12 and 16 MPa, much less than the filed construction cube samples strength of 20 to 25 MPa. But, in spite of this low value, strength was the main cause of the observed structural problems only when:

1 Extra loads were applied in excess of design loads (adding a story or two in addition to the number of stories considered in design).
2 Strength was less than 10 MPa.

The main reason for structural problems was differential settlement of foundations. All roofs were well protected from rain and corrosion was rarely observed.

Out of hundreds of buildings around, a limited number had problems. It can be assumed that most of these buildings have strength of no more than 16 MPa and are still in acceptable condition. As far as, the design loads are not exceeded and no differential settlement occurred. It is known that designers in that area incorporate small service stresses in their designs.

CONCRETE IN A CORROSIVE ENVIRONMENT

Qatar is a semi-island dominated by desert. Iron ores do not exist, and wood does not exist in a scale sufficient to supply the construction in wood. In contrast, limestone is available in large quantities and so is the clay with the quality needed for the manufacturing of cement. Consequently, concrete emerges as the only reliable structural material available locally. Only for special structures do other structural materials prove to be more competitive.

The achievement of high strength concrete is relatively costly because of the following factors:

1 Although good limestone aggregate is available, the properties of the stone layers varies considerably so that a careful selection should be made if the best quality aggregate is to be extracted.
2 Curing is a major problem because of the high temperature and the scarcity of clean potable water.
3 Site control is tough and is not always easy to provide.

On the other hand, durability of concrete has been causing a lot of worry about the efficiency and reliability of the material. Many engineers had become satisfied with limiting the life of buildings to 15 to 20 years. Some argue that the solution comes with importing aggregate from neighboring countries. Many are dragged by the international trend into seeking higher strength properties to solve the prevailing problems.

It has become clear that the main problem for durability is corrosion caused by chloride contamination. However, when good practice is exercised, i.e., selection of good aggregate (type A), using cold water in mixing and efficient curing with clean water for a week, the concrete is good in durability and strength. This is achieved by well established local or foreign companies at a relatively high price. For ordinary commercial and residential building, such a high cost is not acceptable.

An examination of the concrete actually used in ordinary housing projects was necessary to shed a light on the real concrete properties needed.

Concrete in Public Housing in Qatar

Public housing is a government-supported scheme started in 1966 for providing one-story houses for the lower-income population. House cost is set to be limited so that everybody involved does not take the construction of such houses as highly profitable. The average cost per square meter of built area in these houses in 1992 was 980 QR./m^2 (270 US$ / m^2), while the average construction cost of a villa was 2170 QR./m^2.

An investigation of concrete strength and condition in houses constructed under this scheme was carried out in 1994 [1]. Thirty-four houses were selected at different locations in the city of Doha, having different types of designs, different contractors and different sources of concrete. Both destructive and nondestructive methods were used.

The average strength of twenty-five houses under construction was found to be 14.8 MPa with a min. value of 11.7 MPa . For comparison, three private houses in the same areas were investigated and the average strength for these was found to be 25.0 MPa.

It was noticed that contractors who participate in public housing tenders are different from those involved in the building of private houses and villas. Using the same source and grade of ready-mixed concrete, private houses contractors achieved an average concrete strength in the structures which is 1.5 times that achieved by public housing contractors.

Six old houses set for demolition were also examined. They were about 25 years old. The strength was as seen in Table 1 with an average of 13.7 MPa and a minimum value of 9.9 MPa. The chloride & sulphate contents are also shown in Table 1, high values are evident. It was constantly found, however, that corrosion occurred in the roof slab because of rain water penetration. Water proofing was damaged and steel cover was very small. Other parts of the structure did not show signs of corrosion in-spite of the high chloride contamination. Other causes of deterioration were foundations settlement and temperature-caused volume changes.

Table 1 Concrete in 25 years old houses

House No.	Average Compressive Strength MPa	Chloride Content per Weight of Cement %	Sulphate Content per Weight of Cement %
1	10.0	0.82	11.9
2	29.7	1.76	7.1
3	9.9	1.42	5.2
4	10.7	1.14	7.2
5	11.8	0.83	3.0
6	10.3	0.48	6.2

In Qatar, the Ministry of Municipal Affairs supervises the building of both private and public housing. Cube samples from casts in these houses are tested and recorded strengths are kept in files. These files were surveyed in full starting from the year 1978 till 1994. The maximum, average and minimum vales of 28 days strength are found and are shown in Figure 1. Although the average values seem to be good, the minimum values are clearly low and are similar to the values found in the public housing investigated.

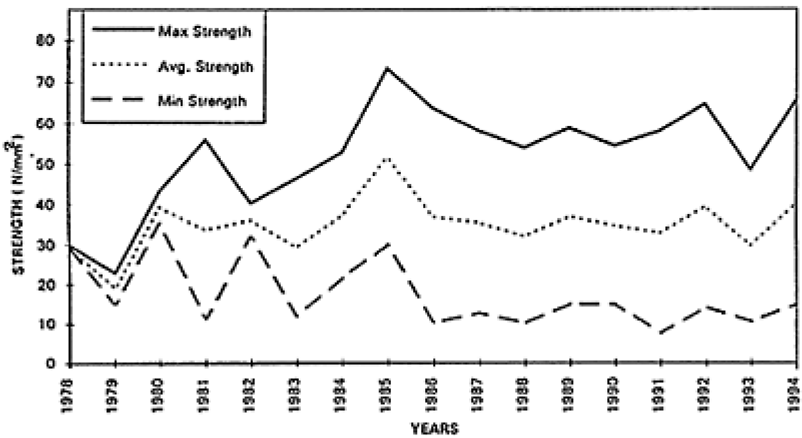

Figure 1 Compressive strength of cubes from sites

How low the strength of concrete in an existing structure could be? Is a question that might be raised having shown the low values found in the mentioned investigations.

Using an Ultrasonic Pulse Velocity tester PUNDIT, a correlation between concrete cube strength and UP velocity was developed over a number of years [2]. The correlation curve is shown in Figure 2. It is suggested that the shown minimum strength is the lowest that can be found in an existing structure.

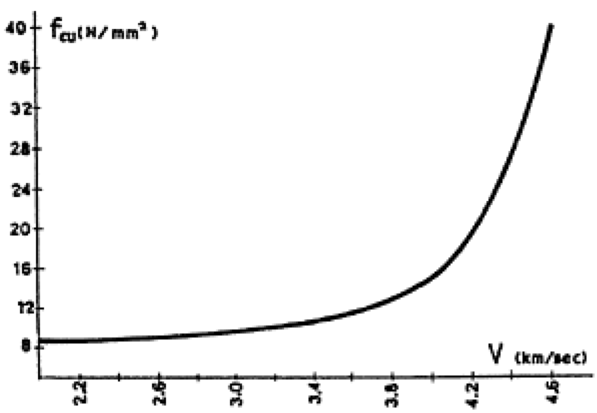

Figure 2U.P.V. versus cube strength (from ref. 2)

DESIGN VERSUS CONSTRUCTION

Having exploited the major cause of deterioration in the severe environment, protection from corrosion should be the major factor to be considered to achieve an acceptable durability. Design plays a role. Knowing that low strength is more likely to be achieved by construction, a practical design should be based on the expected values. Controlling deflection and crack width should be a major objective of design. Eventually a proper protection to steel bars will be accomplished.

To see how design can achieve the mentioned objectives, a design procedure based on the selection of steel reinforcement ratio using the methods of ACI-318 [3] was carried out to design a rectangular beam cross-section for an ultimate applied bending moment M_u= 340 kN.m. Four categories of materials grades are chosen, materials A, B, C and D as shown in Figure 3.

The resulting cross-sections for $\rho=0.50\rho_{max}$ are given in Table 2. The maximum instantaneous deflection versus ratio of reinforcement is plotted in Figure 3. Lower concrete strength will result in a larger cross-section, hence, smaller deflection . Steel strength has the opposite effect because of the design code assumptions.

The calculated average crack widths shown in Table 2 (ACI formula) show that lower concrete and steel strengths reduce the values and proper detailing causes a further reduction. A crack-width reduction of 30% is reached by using a higher percentage of concrete (15% by volume of lower grade concrete) and higher percentage of steel (15 % by area of lower grade steel). The depth increase is not usually a problem in low rise buildings.

Table 2 Design of a Cross-Section

Materials	$\rho=0.50\rho_{max}$	b (mm)	h (mm)	A_s (mm^2)	W_1 3 bars in a row (mm)	W_2 6 bars in a row (mm)	W_3 6 bars, 4 in the first row (mm)
A	0.01204	250	745	2059	0.200	0.177	0.170
B	0.00813	250	780	1457	0.260	0.266	0.218
C	0.01806	250	620	2524	0.203	0.181	0.173
D	0.01219	250	645	1786	0.264	0.231	0.222

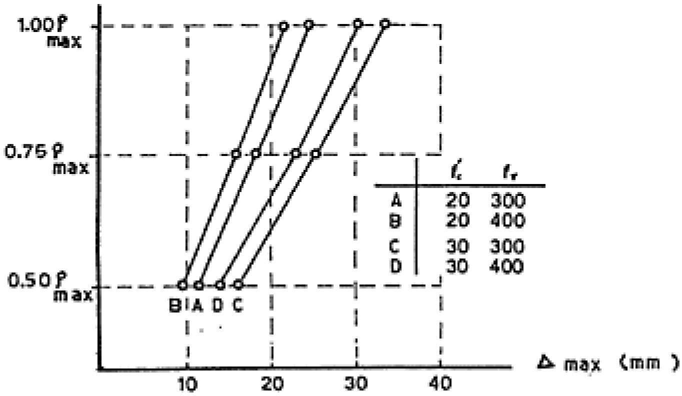

Figure 3 Deflection versus ratio of reinforcement

CONCLUSIONS

1. High concrete strength does not guarantee great durability.
2. In corrosive environment, protection of concrete may be more important for durability than great strength.
3. Foundation stability is very important for durability of structures.
4. Significant differences exist between contractors in concrete quality, even when beginning with the same ready mixed concrete.
5. Design and construction of structures should include recognition of low-strength value concrete in constructing durable structures.

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BEHAVIOUR IN EXTREME CLIMATES OF CONCRETE MADE WITH DIFFERENT TYPES OF CEMENT

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ABSTRACT. The objective of the research programme was to determine the effect of cement type on the behaviour of concrete in extreme climates. The cement types considered were Ordinary Portland Cement and Blended Portland Cements: Portland Pozzolana, Pozzolanic and Blast Furnace Slag Cements. The behaviour of concrete was assessed by the compressive strength development of cube specimens subjected to different curing cycles (temperatures of 5°C, 20°C and 30°C, with curing times of 5 hours, 7 and 28 days). The compressive tests were performed at ages ranging from 1 to 90 days. The observed mechanical behaviour of Portland Cement shows an optimum at a curing temperature of 20°C. However, different behaviours appeared for extreme conditions for Cements with additions. In the latter case, the results indicate that the maximum strength is obtained for a higher curing temperature.

Keywords: Portland Cement, Portland Pozzolana Cement, Pozzolanic Cement, Blast Furnace Slag Cement, Strength, Extreme climates, Hot weather, Cold weather.

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INTRODUCTION

Concrete Technology bases its general recommendations on 20°C concrete temperature and on Portland Cement. However, changes on concrete temperature with respect to the reference temperature generate changes on the behaviour of concrete both on the fresh and hardened state. For this reason temperature requirements for concreting on extreme climates are specified on technical bibliography [1,2]. These constraints are due to the following main effects of temperature on concrete: variation on the amount of water to change slump [1,3], variation of the concrete strength [1,3,4,5] and variation on the strength-maturity relationship of concrete [3,6]. Thus, the understanding of the phenomena that affect the behaviour of concrete at extreme temperatures, low and high, is required.

On the other hand the large use on some countries of Blended Portland Cements requires the knowledge of the influence of temperature on the properties of concrete manufactured with those cements. Therefore, the main aim of work reported in this paper was to examine the behavioural changes of concretes made with different types of cement due to temperature effects. In this way it is expected to provide information allowing to take logical decisions for concreting on site under extreme weather conditions. This information is of paramount importance particularly in the case of infraestructural works (bridges, pavements, etc.) that are commonly exposed to extreme temperatures, both high and low.

EXPERIMENTAL PROGRAMME

Tacking into account the definitions for cold and hot weather concreting given in the Chilean standard [2] and considering the extreme temperatures that are normally encountered on construction jobs, the minimum concrete temperature on cold weather was defined as “5 °C” and the maximum temperature on hot weather as “30 °C”. With these definitions and considering a reference temperature of “20 °C”, a study of the influence of temperature variations on concrete behaviour was carried out from the following points of view: effect of its magnitude (5, 20 and 30 °C) and effect of the exposition period or curing time under those temperatures (particularly, curing times of 5 hours, 7 and 28 days were studied).

Effect of the Temperature on the Behaviour of Concrete with different Types of Cement

This research examines the behaviour of concrete manufactured with four types of cement. The cement types considered were Portland Cement (OPC) and Blended Portland Cements: Portland Pozzolana, Pozzolanic and Blast Furnace Slag Cements. In Chile these cements are commercially available with the additions included.

The composition of these cements was [7]:

- Ordinary Portland Cement (OPC): Clinker and gypsum.
- Portland Pozzolana Cement: Clinker, 28% of pozzolana and gypsum.
- Pozzolanic Cement: Clinker, 32% of pozzolana and gypsum.
- Blast Furnace Slag Cement: Clinker, 60% blast furnace slag and gypsum.

Concrete designed for two water-cement ratios (0.45 and 0.60) were studied (except for pozzolanic cement where only a W/C of 0.60 was analysed). The compressive strength at 1, 2, 3, 7, 28 and 90 days and the slump to all batches were measured.

A total of 21 test mixes were prepared, which included the following test variables in respect to temperature and curing time:

- Cold:
  - – 5 hours at 5 °C, rest of time at 20 °C.
  - – 7 days at 5 °C, rest of time at 20 °C.
  - – 28 days at 5 °C, rest of time at 20 °C.
- Hot:
  - – 5 hours at 30 °C, rest of time at 20 °C.
  - – 7 days at 30 °C, rest of time at 20 °C.
  - – 28 days at 30 °C, rest of time at 20 °C.
- Normal:
  - – Reference: permanently at 20 °C.

The following variables were kept constant:

- Curing humidity (by sealing the samples with double polyethylene bags).
- Water content.
- Type and grading of the aggregate.

Mix Proportions

The mix design was performed according to the 1988 British method [8], adjusted to Chilean conditions. The aggregates were combined to have a grading between curves 2 and 3 of Road Note N°4 [3]. To achieve the different W/C ratios the cement content was adjusted. The mix proportions of the studied concretes are shown in Table 1.

Table 1 Concrete mixes tested

W/C	Cement (kg/m^3)	Water (kg/m^3)	Sand (natural) (kg/m^3)	20 mm crushed Coarse Aggregate (kg/m^3)	40 mm crushed Coarse Aggregate (kg/m^3)
0, 45	400	180	830	295	720
0, 60	300	180	875	310	755

Setting Up, Casting and Testing Procedures

Some important experimental details are briefly described now. All what it is not mentioned is normal laboratory practice.

The concrete was mixed in a 0.1 m^3 capacity rotating mixer. Two 0.08 m^3 batches were required to cast the twenty two 150 mm cubic samples for each concrete mix to be tested. This was done to allow a better statistical analysis. Some procedures were used to achieve the temperatures, and are as follows:

- The concrete constituents were kept at the required curing temperatures for three days before batching.
- The aggregates were measured by weight, considering their humidity.
- The sample were sealed with double polyethylene bags and stored in special curing rooms. The samples were removed from the moulds after 24 hours.

Once the concrete samples were made, they were stored in special curing rooms or freedge at different temperatures. Once the curing time at a given temperature was ended, they were taken to the standard curing room at 20 °C.

The samples were tested in compression using a TONIPAC 3000 testing machine, with special design on the tilting mechanism of the upper platen for testing cubes.

PRESENTATION AND ANALYSIS OF EXPERIMENTAL RESULTS

Influence of Curing Temperature on Compressive Strength of Concretes with Different Types of Cement

A summary of the results of the compressive strength tests performed is found on Table 2 for concretes with different types of cement, water-cement ratios, temperatures and curing times and ages.

Examples of the effect of curing temperature and time on the development of strength of concretes are illustrated in Figures 1 and 2.

The compressive strength versus time curves for a specific type of cement and different curing times intersect each other (Figures 1 and 2). The concrete specimens cured at 5 °C show lower initial strength but their strength is generally higher at 90 days. The concretes cured at 30 °C show the opposite behaviour.

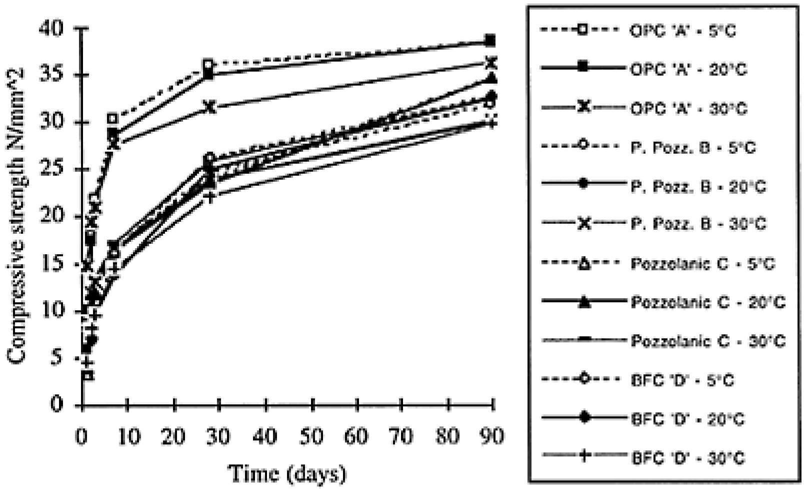

Figure 1 Effect of temperature during the first 5 hours after casting on the development of strength of concretes made with different types of cement and W/C=0.60 (all specimens cured at 20°C after the age of 5 hours)

Table 2 Compressive strength test results for concretes made with different types of cement

CEMENT TYPE	W/C	Curing Temp. (°C)	Curing Time	Strength (N/mm^2) at the age (days) of: 1	2	3	7	28	90
			5 hr.	18, 6	27, 8	30, 4	42, 4	47, 4	54, 4
		5	7 ds.				37, 6	53, 4	59, 6
			28 ds.					47, 5	53, 6
	0, 45	20	28 ds.	23, 0	31, 6	34, 4	43, 5	49, 6	61, 7
			5 hr.	22, 8	29, 5	31, 3	35, 3	44, 1	53, 6
Portland		30	7 ds.				38, 8	43, 3	50, 3
			28 ds.					44, 0	47, 7
“A"			5 hr.	10, 8	18, 0	21, 8	30, 4	36, 1	38, 5
		5	7 ds.				27, 0	39, 3	43, 9
			28 ds.					37, 0	46, 0

	0, 60	20	28 ds.	10, 1	17, 5	20, 6	28, 8	34, 9	38, 6
			5 hr.	14, 8	19, 5	21, 1	27, 6	31, 7	36, 4
		30	7 ds.				29, 6	32, 3	39, 2
			28 ds.					34, 2	38, 0
			5 hr.	6, 4	14, 0	17, 8	23, 9	33, 6	44, 3
		5	7 ds.				16, 9	30, 2	40, 9
			28 ds.					28, 1	41, 8
	0, 45	20	28 ds.	6, 8	14, 2	17, 6	23, 3	28, 6	37, 4
Portland			5 hr.	14, 3	18, 6	20, 7	25, 6	33, 8	42, 7
Pozzolana		30	7 ds.				28, 5	35, 3	40, 9
			28 ds.					40, 2	42, 2
“B”			5 hr.	4, 5	10, 3	12, 7	16, 6	24, 6	32, 0
		5	7 ds.				11, 4	23, 0	31, 6
			28 ds.					18, 8	31, 3
	0, 60	20	28 ds.	6, 0	10, 4	12, 9	16, 4	24, 0	30, 0
			5 hr.	8, 8	11, 9	13, 1	16, 6	24, 0	30, 1
		30	7 ds.				21, 4	26, 2	30, 0
			28 ds.					10, 0	34, 2
			5 hr.	4, 4	0	11, 8	16, 5	24, 0	34, 6
		5	7 ds.				12, 2	23, 7	36, 3
Pozzolanic			28 ds.					19, 8	32, 5
	0, 60	20	28 ds.	3, 8	8, 9	11, 9	16, 5	23, 5	34, 7
“C”			5 hr.	8, 9	11, 2	13, 4	17, 1	26, 0	32, 5
		30	7 ds.				18, 6	26, 9	31, 3
			28 ds.					30, 0	33, 0
			5 hr.	8, 8	14, 6	18, 1	25, 3	41, 6	52, 0
		5	7 ds.				15, 4	38, 6	51, 1
			28 ds.					25, 6	49, 8
	0, 45	20	28 ds.	8, 7		18, 4	24, 1	35, 7	46, 6
Blast			5 hr.	11 ,4	14, 3	16, 4	24, 3	40, 8	46, 8
Furnace		30	7 ds.				32, 7	39, 3	46, 0
Slag			28 ds.					47, 8	52, 8

			5 hr.	4, 4	8, 0	10, 1	16, 2	26, 3	32, 7
“D”		5	7 ds.				8, 3	27, 0	35, 1
			28 ds.					16, 7	34, 2
	0, 60	20	28 ds.	4, 9	7, 1	9, 5	13, 9	25, 0	32, 8
			5 hr.	4, 6	8, 1	9, 6	14, 6	22, 1	29, 9
		30	7 ds.				22, 0	28, 5	33, 0
			28 ds.					30, 6	34, 6

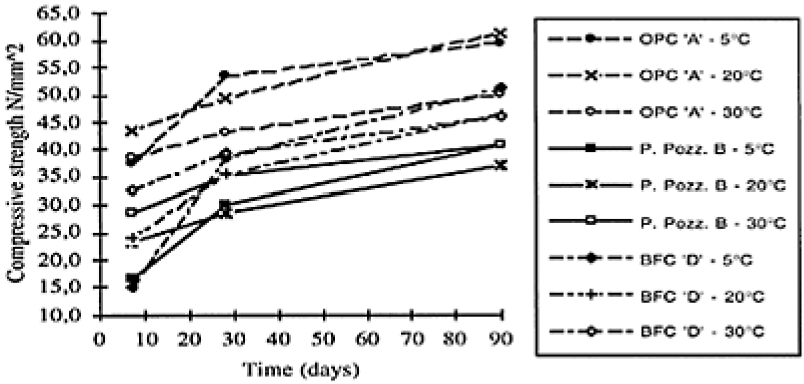

Figure 2 Effect of temperature during the first 7 days after casting on the development of strength of concretes made with different types of cement and W/C=0.45 (all specimens cured at 20°C after the age of 7 days)

In Figures 3, 4 and 5 the relative compressive strength (ratio of strength of concrete cured at different temperatures and ages to the strength of concrete cured at 20 °C) versus time are shown.

In general, for all cement types a tendency to have higher long term compressive strengths when cured at 5 °C than cured at 30 °C is observed. This can be clearly seen in the relative compressive strength charts (see Figure 3). For slower strength gain cements this effect is very noticeable, as can be seen comparing Figures 4 and 5. With these cements the curves intersect and at an age of 90 days the strength of samples cured at 5

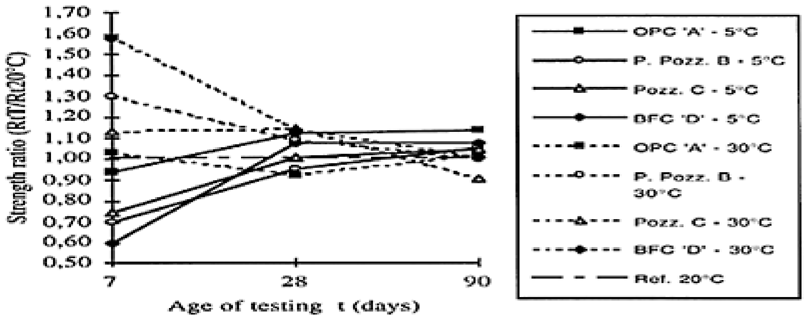

Figure 3 Ratio of strength of concrete cured at different temperatures to the strength of concrete cured at 20 °C (W/C=0.60; the specimens were cured at the indicated temperature during the first 7 days after casting, then cured at 20°C)

°C have surpassed the ones cured at 20 °C. The samples cured at 30 °C have similar or higher strengths, but the rates of strength gain are descending. High early strength cements show a similar tendency, but less marked. For OPC cements the same effect can be seen (Figure 5).

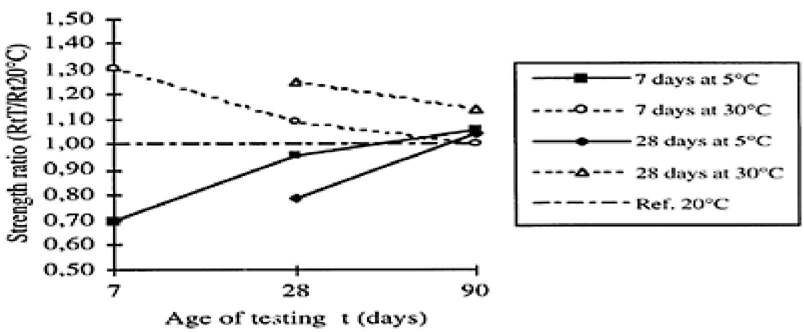

Figure 4 Ratio of strength of concrete cured at different temperatures to the strength of concrete cured at 20 °C (Portland Pozzolana cement 'B' and W/C=0.60; the specimens were cured at the indicated temperature and days after casting, then cured at 20 °C)

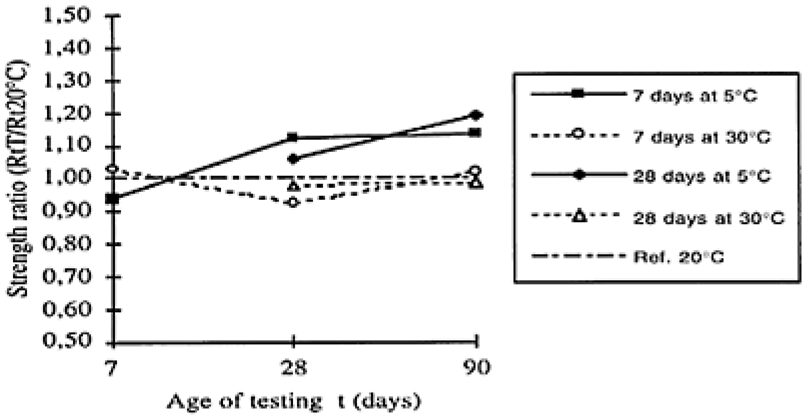

Figure 5 Ratio of strength of concrete cured at different temperatures and ages to the strength of concrete cured at 20 °C (Ordinary Portland Cement 'A' and W/C =0.60; the specimens were cured at the indicated temperature and days after casting, then cured at 20 °C)

Effect of Curing Temperature up to the Age of Testing on Concrete Compressive Strength

The effect of the curing temperature up to the test age was also analysed. For concrete samples cured for 5 hours, at different temperatures, the tests were performed at 24 hours. Examples of these results can be seen in Figures 6, 7 and 8 for OPC, Portland Pozzolana and Blast Furnace Slag Cements, respectively.

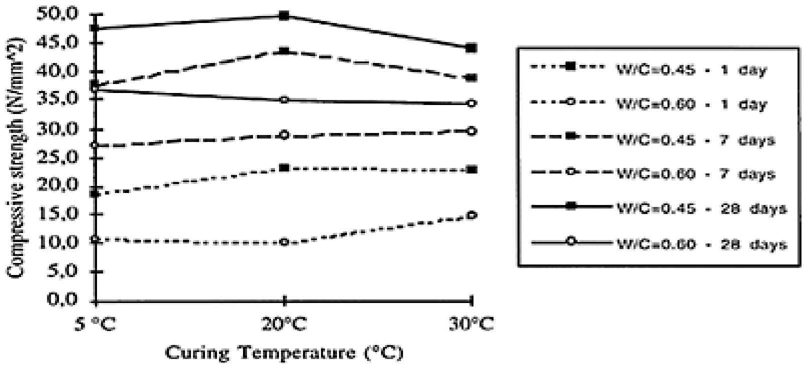

Figure 6 Effect of the curing temperature up to the test age on the strength of concretes with different water cement ratios (Ordinary Portland Cement 'A'; the specimens were cured at the indicated temperature and days before testing)

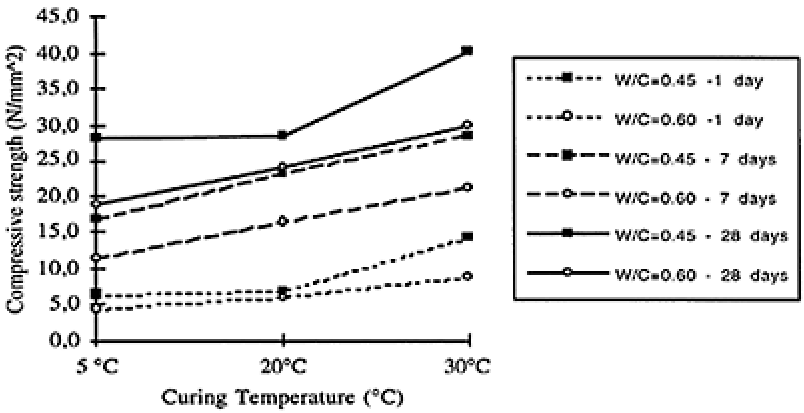

Figure 7 Effect of the curing temperature up to the test age on the strength of concretes with different water cement ratios (Portland Pozzolana Cement 'B'; the specimens were cured at the indicated temperature and days before testing)

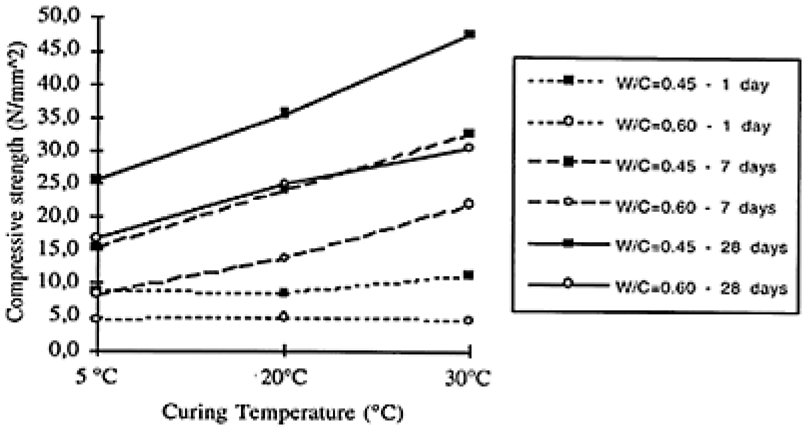

Figure 8 Effect of the curing temperature up to the test age on the strength of concretes with different water cement ratios (Blast Furnace Slag Cement D; the specimens were cured at the indicated temperature and days before testing)

It can be observed in these figures that blended cements show ascending curves with temperature; i.e. the compressive strength is higher the higher the temperature, even for curing time of 28 days. This behaviour is different to what is shown in the technical literature about concrete. For OPC the effect is different (Figure 6); in this case the tendency is similar up to curing time of 7 days, but for concrete samples cured at a given temperature up to 28 days, the higher compressive strengths are found for lower curing temperatures. This behaviour agrees with what is stated in the literature.

This is a very important difference in behaviour between concrete with OPC and Blended Portland Cements, and which this research helped to show. The latter type of cements are the most widely used in Chile. The authors also think that this effect is also valid for concrete mixes with PFA, and other types of pozzolanic additions.

Effect of Curing Temperature on the Strength-Maturity Relation of Concretes with Different Types of Cement

The influence of the curing cycle on the compressive strength—maturity relationship is shown in Figures 9 and 10, for different cement types.

In general, it can be seen that the relationship depends on the type of cement (Figure 9), the water/cement ratio and the curing temperature (Figure 10). Therefore, the relation requires to be determine for each specific concrete, trying to simulate the actual curing conditions on site (i.e. temperature history).

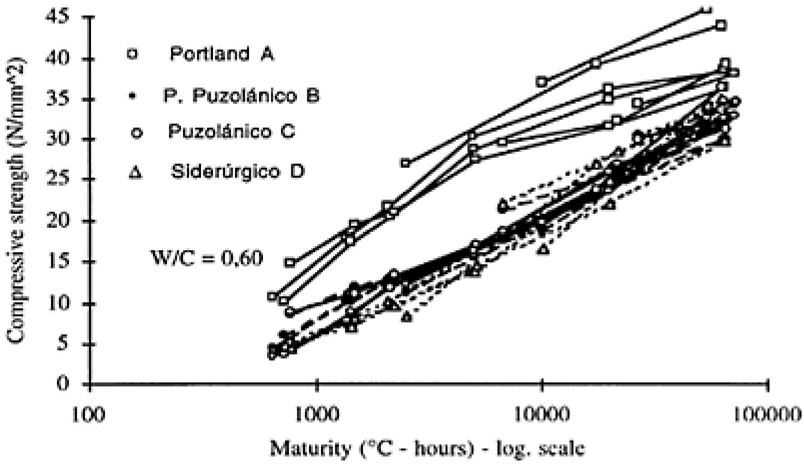

Figure 9 Effect of the type of cement on the strength—maturity relationship

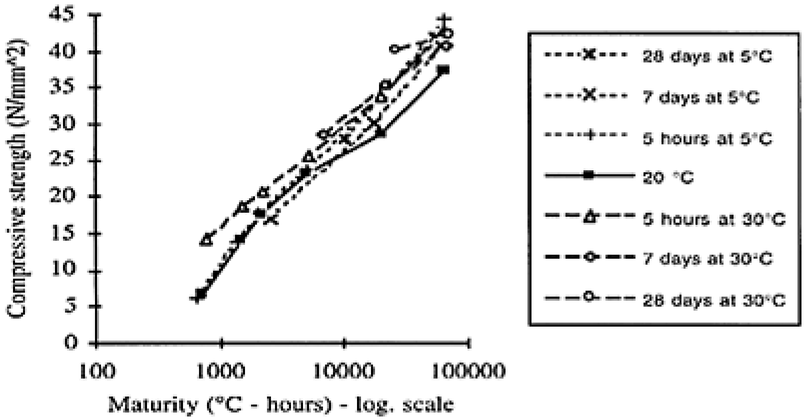

Figure 10 Effect of the curing cycle on the strength—maturity relationship for Portland Pozzolana Cement "B" (W/C=0.45)

The strength-maturity curves for cements with additions show different slopes depending on the curing temperature (Figure 10); i.e. the lower the curing temperature the larger the slope of the strength-maturity relation of the concrete and for maturities larger than approximately 20.000 °C-hour the concrete present higher strengths when cured at low temperatures than cured at high temperatures. For maturities smaller than 10.000 °C-hour the opposite behaviour is observed. It is important to note that concretes cured at 30 °C during 28 days show the smallest slope (Figure 10). For OPC the effect of the temperature on the strength-maturity relation is the same but the benefit of lower curing temperatures is more noticeable even for smaller maturities (Figure 9).

Effect of the Type of Cement on Freezing Temperature of Concrete

The freezing temperature of fresh concrete was also studied, for concretes made with 3 types of cement (OPC, Portland Pozzolana and Blast Furnace Slag Cements) and 2 water/cement ratios (0.45 and 0.60). The concrete and water freezing temperatures were determined from the cooling curves obtained when the materials were subjected to a constant cooling environment. The concrete specimens to be tested were cast in 70 mm cubes. The concrete samples were introduced immediately after casting in a freezer maintained at a constant temperature of ?20 °C. The evolution of the fresh concrete temperature was measured every 3 minutes, during a period of 165 minutes after casting, by means of termocouples inserted in the fresh concrete and connected to a digital temperature instrument. The measured results are show in Figure 11.

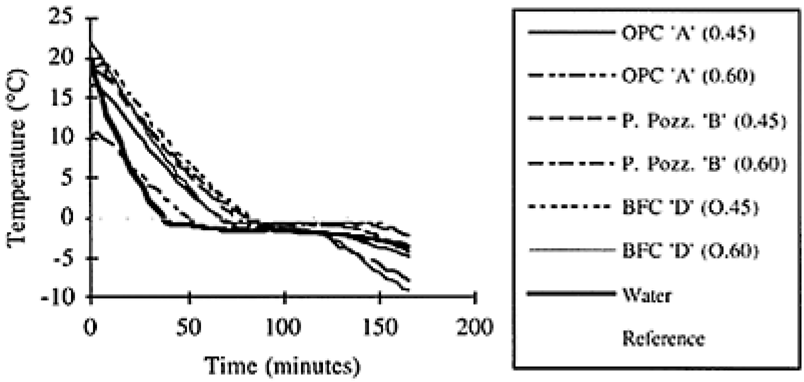

Figure 11 Effect of cement type and W/C ratio on freezing temperature of concretes (W/C ratio in brackets)

Figure 11 shows that the freezing temperature of concrete and water were of the order of −0,4 and −1,5 °C and fresh concrete freezes at approximately the same temperature as water independent of the type of cement used. Therefore, it was apparent that the soluble salts of the cement do not alter the freezing point of the water in the concrete, as it was expected. The results also show that the cooling rate of fresh concrete is independent of the type of cement used, but is smaller than the cooling rate of water alone. This is probably due to the test conditions and because at very early ages the cements do not liberate a sufficient amount of heat allowing to differentiate its cooling rates. Figure 11 shows that a decrease of the W/C ratio delays the time the concrete takes to freeze (probably because of the larger proportion of cement contained in a concrete with a smaller W/C ratio), but the freezing temperature does not change.

CONCLUSIONS

The observed relation between curing temperature and mechanical behaviour of concrete made with Portland Cement agrees well with results reported on technical literature. However, for cements with additions the relation is different.

It was concluded that for Portland Cement the effect of the curing temperature on the compressive strength shows an optimum at 20 °C. For Blended Cements, the results appears to indicate that the maximum strength is obtained for a higher curing temperature. In the latter case, a higher curing temperature gives a higher compressive strength for the same maturity, both for short and long term. The results are similar for the two water/cement ratios analysed (0.45 and 0.60).

The maturity-compressive strength relations are lineal in a semi-logarithmic chart and are particular for each concrete and cement utilized.

It was shown that fresh concrete freezes at the same temperature as water and that the minimum temperature the concrete can reach is 0 °C. Therefore, the minimum temperature allowed for the fresh concrete in cold weather construction should be 5 °C, before applying the protection needed (insulation or heat). This will allow to obtain higher long term compressive strengths and to avoid concrete freezing. Also, it should be avoided to saturate the concrete during the early life; curing procedures should be directed to prevent the loss of water by evaporation rather than wetting.

For hot weather concrete construction, the results show that the concrete placing temperature should be as low as possible and the convenience to maintain the concrete at low temperature during the early life of the concrete.

ACKNOWLEDGEMENTS

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PROBLEMS ABOUT OPTIMIZATION OF POROSITY AND PROPERTIES OF AERATED CONCRETE

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Appropriate Concrete Technology. Edited by R K Dhir and M J McCarthy. Published in 1996 by E & FN Spon, 2–6 Boundary Row, London SE1 8HN, UK. ISBN 0 419 21470 4.

ABSTRACT. The mechanical and thermal properties of aerated concretes depend directly on the porosity. The quantity and the geometrical configuration of pores depend on the quantity and the nature of the constituents which enter into the composition of the material. The manufacturing process leads to the obtention of anisotropic materials which are constituted by uniaxially flattened pores, which are perpandicular to the direction of the expansion. This anisotropy may be an advantage for the optimisation of mechanical and thermal properties of aerated concretes. This study shows the lightening phenomenon and the relation existing between the quantitative and qualitative aspects of porosity. Moreover this study shows that the phenomenon of mechanical anisotropy depends on relativity of kinetics of setting and expansion which seems to be a fundamental parameter to optimize properties of aerated concretes.

Keywords: Aerated concretes, Optimization, Introduced porosity, Compressive strengths, Mechanical anisotropy, Kinetic of setting, Kinetic of expansion .

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INTRODUCTION

Generally the mechanical and thermal properties of porous materials, and particularly of aerated concretes, depend directly on porosity. In the case of aerated concretes porosity is created by a chemical process[1]. It consists in adding aluminium powder to the constituents of the mortar (cement, lime, sand and water), which produces a gazeous release which causes fresh mortar to expand. The quantity and the geometrical configuration of pores depend on the quantity and on the nature of the constituents which enter into the composition of the material [2].

The manufacturing process leads to the obtention of anisotropic materials which are constituted by uniaxially flattened pores which are perpendicular to the direction of the expansion. This anisotropy may be an advantage for the optimisation of mechanical and thermal properties of aerated concretes[3] [4].

The primary aim of this work is to study the lightening phenomenon and the influence of the introduced porosity on the mechanical anisotropy of the aerated concretes. The introduced porosity depends on the parameters of composition which also act on relativity of setting kinetic and expansion kinetic; the second aim of this study is to correlate the relativity of setting and expansion kinetics with introduced porosity and mechanical anisotropy.

INFLUENCE OF INTRODUCED POROSITY ON COMPRESSIVE STRENGTHS.

For autoclaved aerated concretes, the autoclavage improves the mechnical strengths of the matrix but it takes place in the process after the creation of the pores and it does not have any effect on the introduced porosity. Thereby lightening phenomenon and mechanical anisotropy due to geometry of pores are the same for autoclaved or not autoclaved aereted concretes though mechanical strengths are different.

The principle of the study consists in the realisation of different not autoclaved aerated concretes with different compositions and the corresponding basic matrix. We have quantified the introduced porosity (PI) with apparent densities of expanded mixtures (MVAe) and corresponding apparent densities of the basic matrix (MVAb): PI=(MVAb-MVAe)/MVAb. The compressive strengths (RC) have been measured in the two perpendicular directions to display the mechanical anisotropy. The compositions of the basic matrix are given in the table. 1. The ratios of cement (C), lime (X) ,water (E) and sand (S) are expressed comparatively with the weight of binder (L). For aereted concretes the aluminium powder (AL) quantities added to basic matrix and expressed by ratios comparatively with the weight of binder are : 0.01; 0.025; 0.05; 0.1; 0.2; 0.3; 0.4; 0.5; 0.75; 1; 1.5; 2.

Tab.1 Compositions studied for basic matrix

composition	C/L	X/L %	E/L%	S/L%
C3	0	100	45	0
C6	100	0	40	20
C7	100	0	40	40

Figure 1 shows the evolution of compressive strengths in accordance with the introduced porosity. The compressive strengths are noted RCij ; the indice (i) corresponds to the direction of the measure and the indice (j) to the composition. The evolution of compressive strengths in accordance with porosity shows that the strengths measured in the direction of the expansion of the material are lower than the strengths measured in the perpandicular direction.

This difference of strengths which shows the mechanical anisotropy begins for the first values of the porosity and the difference increases with porosities between 0.3 and 0.45

For the porosities greater than 0.45 the difference decreases and the material tends to be isotropic.

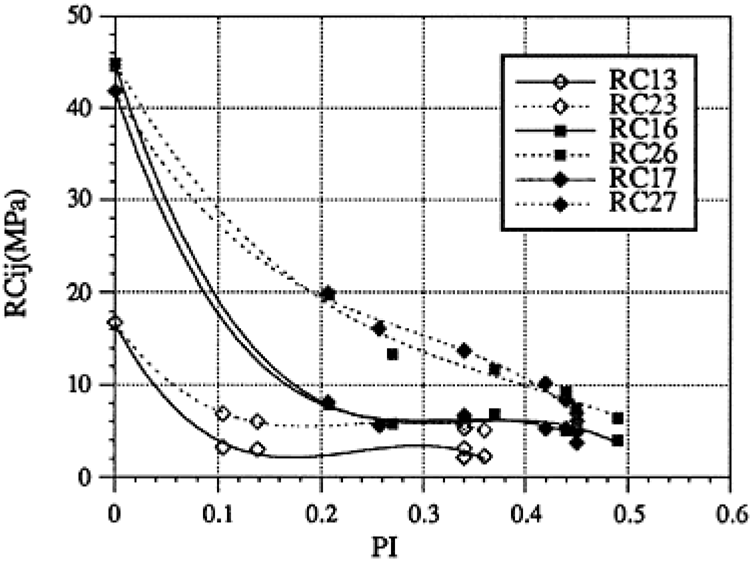

Figure. 1: Evolution of compressive strengths in accordance with the intoduced porosity

INFLUENCE OF THE RELATIVITY OF KINETICS OF SETTING AND EXPANSION ON THE MECHANICAL ANISOTROPY

During the manufacturing process of aerated concrete, we observe a swelling of material which is due to the formation of pores in the mortar. This formation of pores is caused by the expansive agent. The formation of pores depends on the degree of hardening of the mortar, therefore on the setting kinetic, and on the gazeous release speed, therefore on the expansion kinetic. The mechanical anisotropy observed in the material and due to the flattening of pores should have a relation with the relativity of kinetics of setting and expansion; indeed a quick setting and a slow gazeous release prevent the expansion of material and causes the flattening of the pores. On the opposite, with a quick swelling and a slow setting, the expansion of the material is easier and the pores tend to be more spherical. We have tried to show this phenomenon in this part of the study.

Methodology

In order to characterize the kinetics of setting and expansion, we have measured the height of swelling (G), and evaluated the setting by the driving (E) of the Vicat's needle (d=13mm), in accordance with the time for some compositions which have the same matrix but different expansive agent dosages. To characterize the relativity of the two kinetics, we have considered the time TO corresponding with the point of intersection of the two curves obtained. After that, we have investigated for two correlations: the first is between the mechanical anisotropy characterized by the ratio RC2/RC1 and the relativity of kinetics of setting and expansion characterised by the time TO; the second is between the introduced porosity and the relativity of kinetics of setting and expansion.

Results and comments

The results concern the composition C3.

The figures 2,3,4,5, show the kinetics of setting and expansion, and permit to characterize the time TO corresponding to each dosage of the expansive agent.

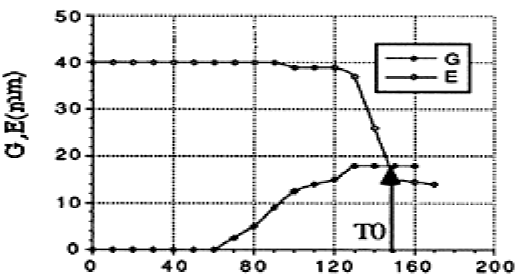

Figure 2: Kinetics of setting and expansion, AL/L=0.3%

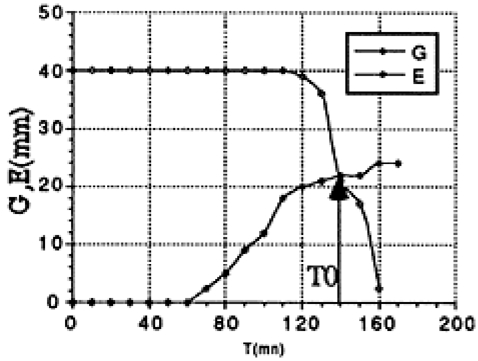

Figure 3: Kinetics of setting and expansion, AL/L=0.4%

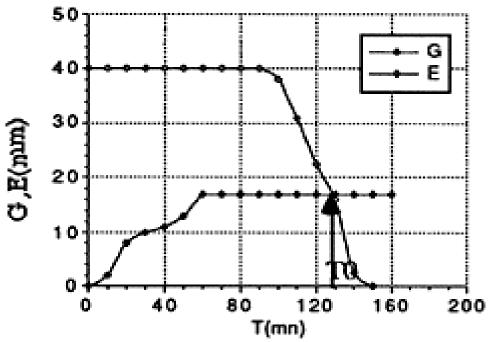

Figure 4: Kinetics of setting and expansion, AL/L=0.75%

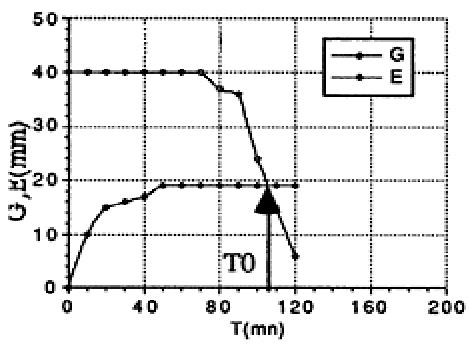

Figure 5: Kinetics of setting and expansion, AL/L=1%

On figure 6 we see that the introduced porosity increases very quickly for the weak expansive agent dosages, and it is stabilized for dosages about 0.75 %. The figure 7 shows that the mechanical anisotropy increases also with the expansive agent dosage but it decreases for the expansive agent dosages greater than 0.75%.

Figure 8 shows the relativity of kinetics of setting and expansion in accordance with the expansive agent dosage.

The results obtained show that when the expansive agent dosage increases, the time TO characterizing the relativity of kinetics of setting and expansion decreases. This phenomenon is explained by the swelling speed which increases when the expansive agent dosage increases. The figures 9,10 respectively show the evolution of the introduced porosity and of mechanical anisotropy in accordance with the relativity kinetics of setting and expansion.

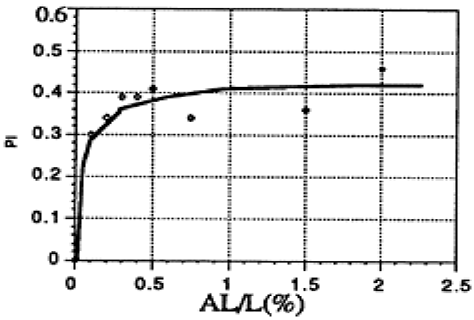

Figure 6: Evolution of the introduced porosity in accordance with the expansive agent dosage.

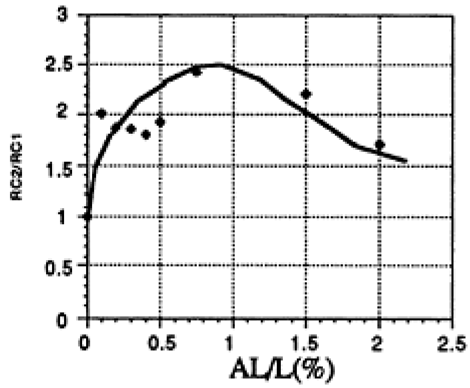

Figure 7: Evolution of the mechanical anisotropy in accordance with the expansive agent dosage.

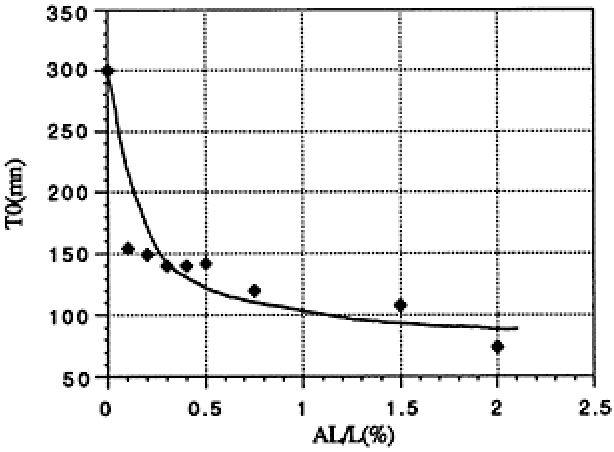

Figure 8: Evolution of the relativity of setting and expansion kinetics in accordance with the expansive agent dosage

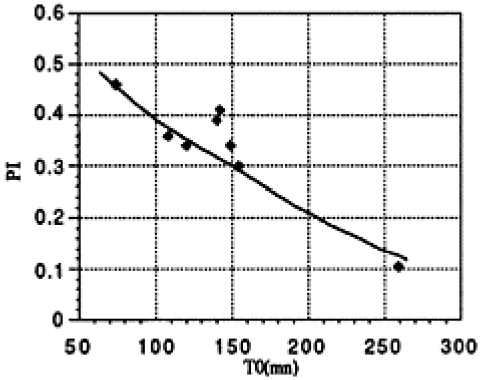

Figure 9: Evolution of the introduced porosity in accordance with the relativity of setting and expansion kinetics

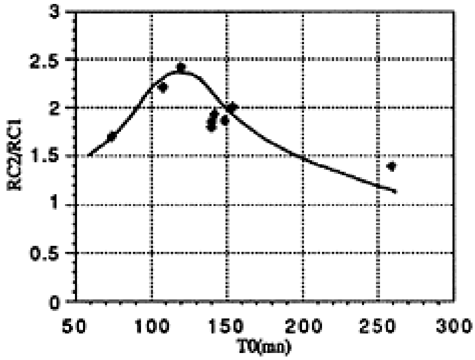

Figure 10: Evolution of the mechanical anisotropy in accordance with the relativity of setting and expansion kinetics

The evolution of introduced porosity in accordance with the time T0 shows that when it increases the introduced porosity decreases linearly. This phenomenon is due to the slowness of the expansion comparatively with the quickness of setting and that prevens the development of the porosity. Concerning the mechanical anisotropy characterized by the ratio RC2/RC1 we observe an optimal time T0 of about 120mn which permits to obtain for this composition a maximal anisotropy.

We observe that the kinetics relativity which is according to the maximal anisotropy doesn't give the maximal porosity.

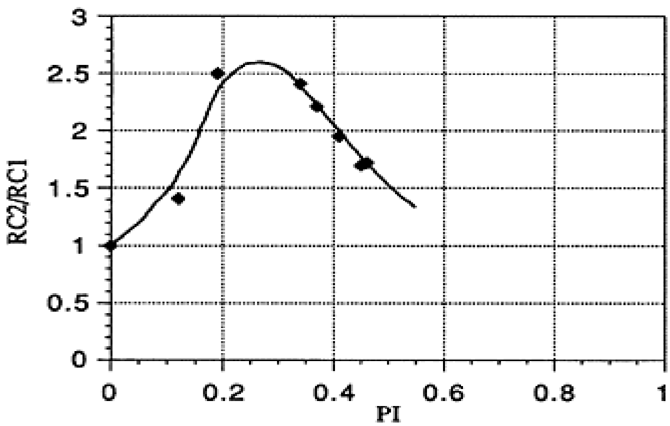

Figure 11: Evolution of the mechanical anisotropy in accordance with the introduced porosity

Indeed the evolution of the mechanical anisotropy in accordance with the porosity (Figure 11) shows that the maximal anisotropy is obtained for porosities about 0.3.

For greater porosities, the anisotropy decreases and the material tends to be isotropic. Finally the introduced porosity and the mechanical anisotropy in aerated concretes are very dependent and their simultaneous optimization requires the mastery of the relativity of kinetics of setting and expansion.

CONCLUSIONS

This study clearly showed the lightening phenomenon which is shown by the evolution of mechanical properties in accordance with porosity. In the same way it showed the relation existing between the quantitative and qualitative aspects of the introduced porosity; indeed the maximum mechanical anisotropy due to the flattening of the pores is obtained for porosities between 0.2 and 0.3. Moreover we established that the phenomenon of the mechanical anisotropy depends effectively on the relativity of kinetics of setting and expansion and that there is an optimal value of this one which permits to obtain a maximal anisotropy. On the other hand this optimum does not give the maximal porosity. This result is in accordance with the evolution of the mechanical anisotropy with the porosity. The results obtained in this study have highlighted the importance of the parameter T0. This time which characterises the relativity of kinetic of setting and expansion determines the developement of the porosity and its geometrical configuration, and seems to be a fundamental parameter to obtain materials more performant from a mechanical and thermal point of view.

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THE USE OF FOAMCRETE FOR AFFORDABLE DEVELOPMENT IN THIRD WORLD COUNTRIES

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Appropriate Concrete Technology. Edited by R K Dhir and M J McCarthy. Published in 1996 by E & FN Spon, 2–6 Boundary Row, London SE1 8HN, UK. ISBN 0419 21470 4.

ABSTRACT. The objective of this paper is to discuss methods of optimizing the mix design of foamcrete as far as strength and cost is concerned, while minimizing the adverse aspects, such as shrinkage, to acceptable levels. Values obtained from experimental work for different design parameters will be evaluated and possible uses in the Southern African market will be discussed.

Foamcrete consists of a cement-based slurry that is aerated by adding a pre-foamed protein based foaming agent. Foamcrete with between 20% and 60% foam in the matrix is currently being investigated. Ungraded pulverized-fuel ash as a cement replacement is available at low cost in large areas of South Africa, leading to tests on the use of ash in foamcrete. As the aim of the investigation is to produce foamcrete products that can be manufactured by rural communities, it is essential to determine how sensitive the results are to small changes in the mix.

A small low cost housing unit, using tilt-up construction, has been developed. Experiments are currently underway for developing roof tiles, manhole rings, drainage channels, meter boxes, coolrooms, fire walls, wall units and building blocks. All these products could be used in the Reconstruction and Development Program currently underway in South Africa.

Keywords: Foamcrete, Pulverized-fuel ash (PFA), Condensed Silica Fume (CSF), Lightweight concrete, Affordable housing, Water/cement ratio, Ash/cement ratio, Protein foam, Low cost infrastructure.

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applications and the use of fiber reinforcing and lightweight concrete for affordable development.

INTRODUCTION

According to the White Paper on Housing “South Africa is characterized by large scale unemployment in the formal sector of the economy. During 1995 40% of South African households earned less than R800/month (approximately £ 150). A relatively small formal housing stock, slow and progressively decreasing rates of formal and informal housing delivery have resulted in a massive increase in the number of homeless people.” [3]

It is estimated that the urban housing backlog in 1995 was approximately 1, 5 million units. Due to the high rate of population growth and the low rate of housing provision, it is estimated that the housing backlog is presently increasing at a rate of 178 000 units per annum [3].

The Reconstruction and Development Program currently underway in South Africa aims at giving all South Africans access to “A permanent residential structure with secure tenure, ensuring privacy and providing adequate protection against the elements” and “potable water, adequate sanitation and domestic electricity supply” [3].

In order to address the needs of the country, modern specialized technology could be used to improve the quality of building materials used in under developed areas. By manufacturing lightweight building blocks, wall panels, roofing sheets, footings and floor slabs on the outskirts of undeveloped areas, the local communities could use their own transport to obtain these building materials. Lightweight aggregate or Autoclaved Aerated Concrete could be used to meet this need, but the cost of the aggregates and the process exclude these possibilities. Foamcrete however is an affordable, transportable building material.

Little has been published on the structural behaviour of foamcrete. Since 1992 tests on the structural properties of foamcrete have been conducted at the University of Pretoria. The effect of foaming agents, percentage foam and mix proportions of other constituents are being investigated.

The objective of this paper is to discuss methods of optimizing the mix design of foamcrete as far as strength is concerned, taking cost into account, while minimizing the adverse aspects, such as shrinkage, to acceptable levels. Values obtained from experimental work for different design parameters will be evaluated and possible uses in the Southern African market will be discussed.

MANUFACTURING REQUIREMENTS

Foamcrete consists of a cement-based slurry that is aerated by adding a pre-foamed protein based foaming agent [7]. The cement paste starts setting around the foam bubbles and when the foam starts degenerating after approximately three quarters of bubbles and when the foam starts degenerating after approximately three quarters of an hour, the paste has sufficient strength to maintain its shape around the void. The density of the foamcrete is a function of the volume of foam that is added to the slurry. To ensure that the desired percentage air is entrained in the mix, pre-foaming, where the foaming agent is aerated before being added to the mix, is recommended.

During the experiments it was found that the foamcrete is sensitive to the type of mixer and the mixing speed used. When mixing foamcrete it is recommended that the foaming agent be added last. The mixing process should be a “soft” process where the foam is gently mixed in, to prevent the foam from breaking down when mixed vigorously.

The density of the foam and the slurry are so different that it is difficult to obtain a uniform foamcrete mix unless a mixer that forces integration is used. The time required for mixing should be reduced to a minimum as the foam has a limited lifetime where after it starts degenerating. If the mixing is too slow the cement paste will not have sufficient curing time before the foam breaks down leading to insufficient strength for maintaining the shape of the void formed when the foam breaks down.

MIX DESIGNS

There is currently no standard method available for designing a foamcrete mix. Foamcrete is more sensitive to water content than normal concrete. When designing the foamcrete mix great care has to be taken, to ensure a homogeneous mix without foam breakdown. Before an engineered product can be designed it is essential to obtain design parameters. These parameters can be established once the material properties have been determined.

Foamcrete with between 20% and 60% foam in the matrix is currently being investigated. The strength of foamcrete is an inverse function of the percentage foam, and therefore the density of the material as indicated in Figure 1. The material currently under investigation has densities ranging from approximately 800 kg/m^3 to 1700 kg/m^3, and only limited tests have been conducted on materials with lower densities.

From the outset of our investigation it was obvious that the mix design procedure and material properties of normal concrete could not be used for foamcrete. Concrete normally has a certain water demand to obtain a required workability and the strength of the mix decreases as the cement content decreases or the water/cement ratio increases. If the water in the foamcrete mix is not sufficient for the initial reaction of the cement, the cement withdraws water from the foam causing rapid degeneration of the foam. If too much water is added, segregation which causes variation in density, tends to take place.

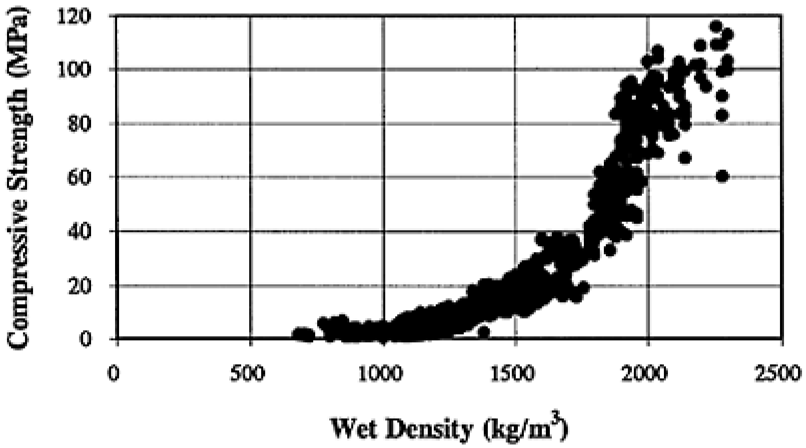

Figure 1: Relationship between Compressive Strength and Density

Compressive strength, modulus of rupture and shrinkage tests have been conducted to determine material properties. Compressive tests were conducted on 100 mm cubes after 7 and 28 days using a special cube press giving readings to the nearest tenth of a kN. Modulus of rupture tests are conducted on 100 x 100×500 mm beams in a closed loop continuous measuring system. The shrinkage readings were taken on 250 mm shrinkage beams. To eliminate temperature effects all specimens were kept under plastic covering at 22 °C and 60% humidity in a constant temperature room for 24 hours where after curing took placed in constant temperature water baths.

To determine the effect of the foaming agent, tests where conducted on samples consisting of only cement, water and foam The water demand for different percentages of foam was obtained by slowly increasing the water/cement ratio to the point where no visual breakdown of foam took place. This ratio was used as a basis and a range of mixes both dryer and wetter than this mix was tested. For each of the foam contents an optimum water/cement ratio, resulting in the highest strength, was obtained [9].

Cost is a major concern in the development of any infrastructure. The aim when designing a foamcrete mix was therefore to optimize strength while minimizing cost. Ungraded ash (PFA) is available at low cost in large areas of South Africa as a cement replacement, which led to further tests on the use of ash in foamcrete [2].

Using mixes with fixed percentages of foam and ash/cement ratios, the optimum water demand was determined as before. The optimum water/cement ratio for various percentages of foam is shown as a function of ash/cement ratio in Figure 2. From Figure 2 it can be seen that the optimum water/cement ratio for all the different foam contents is in the region of 0.38 when no ash is used. The required water/cement ratio increases as the ash/cement ratio increases. This relationship is basically linear for low percentages of foam, but with high percentages of foam the water demand increases to the extent where it is not possible to prepare a durable mix with 60% foam and an ash/cement ratio of 3.

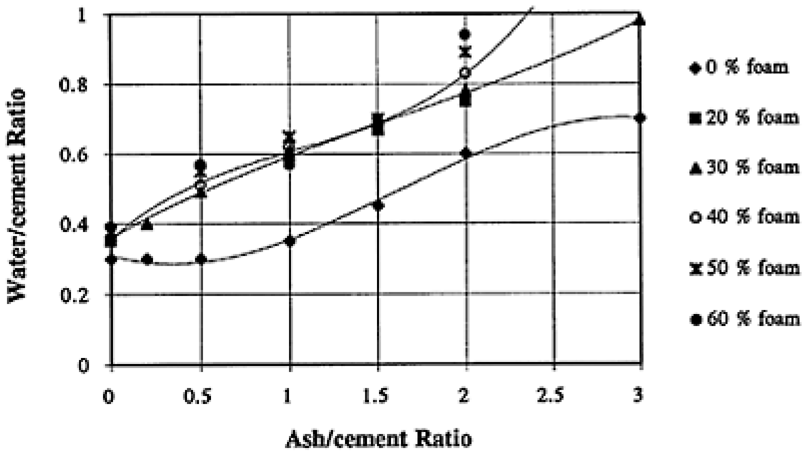

Figure 2: Water/cement versus Ash/cement Ratio

The effect of slagment, chalk, sand, silica fume and superplasticizer on the water demand of mixes with different percentages of foam was determined. The results obtained from these tests are, to the extent that the limited space allows, discussed in the succeeding paragraphs.

STRUCTURAL PROPERTIES

According to Fulton's Concrete Technology (1994) curing in humid conditions under high temperatures result in high early strength for normal concrete [1], but does it necessarily hold true for foamcrete? It has been well documented that the shrinkage of aerated concrete can be reduced by autoclaving [6], but if this method of curing is not affordable does it render aerated concrete useless? Can a foamcrete mix not be designed to optimize strength, while limiting shrinkage and cost? The author has conducted various tests to determine what the effect of mix design and curing would be on the strength and shrinkage of foamcrete.

Compressive Strength

To obtain results unaffected by seasonal variation in temperature, all specimens made at the University were normally cured in water. The results used in this paper are average values and not characteristic values (f_c). No trends in standard deviation was observed.

One would normally assume that wet curing would result in optimum strengths. However tests conducted at our laboratory indicate that this does not hold true for foamcrete as can be seen in Figure 3. Using cubes from one mix, groups where placed in a constant temperature room at 22°C in the open and in plastic bags, in water at 22°C, in plastic bags and open in the oven at 50°C. Cubes were removed from the oven after 2 and 7 days and placed in the constant temperature room for the remainder of the time. Some of the cubes cured under water, were dried for 24 hours before crushing to compare dry results. From these results it can be seen that it would be beneficial to cure foamcrete products under plastic covers rather than having to use water.

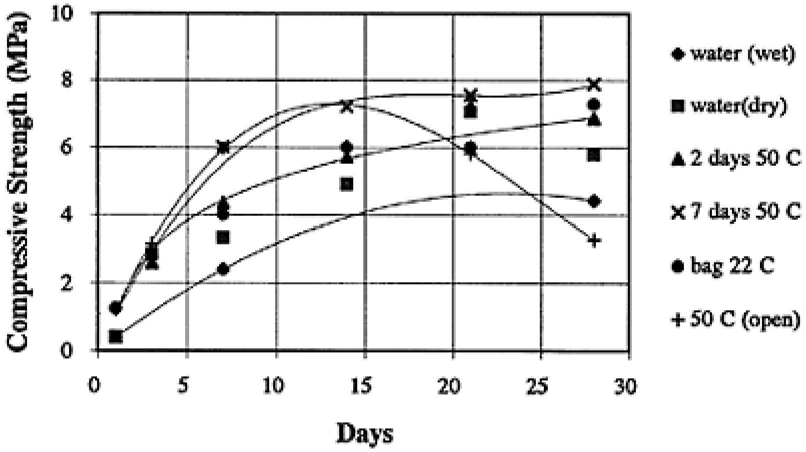

Figure 3: Effect of Curing on Compressive Strength

When plotting Compressive strength as a function of ash/cement ratio the compressive strength decreases as the ash/cement ratio increases for mixes without foam. Where foam is added to the mix, the compressive strength, as indicated in Figure 4 does not decrease with an increase in ash. Results obtained using 30% and 40% foam indicates an optimum in compressive strength at an ash/cement ratio of 1. We thus decided to use an ash/cement ratio of 1 and find an alternative filler to reduce the cost of the mix even further.

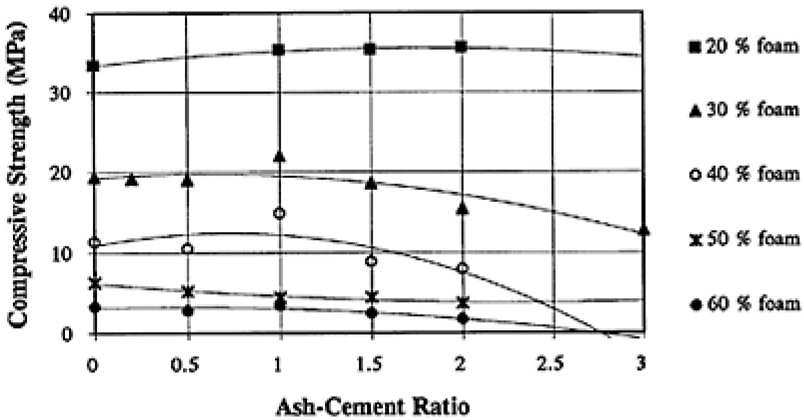

Figure 4: Effect of Ash/Cement Ratio on Compressive Strength

Various materials were used with greater and lesser degree of success but eventually we decided to use sand as an additional filler. Sand with a moisture content varying between 6% and 14% was initially used without drying. This moisture in the sand was sufficient not to have to change the water demand of the mixes. The exact water demand of the mixes were determined by using oven dried sand.

Some of the results obtained when mixing various sand/cement ratios at different foam contents are shown in Figure 5. From these results it can be seen that the compressive strength is significantly reduced by adding sand. It is interesting to note that there is no further significant strength reduction with further increases in the sand/cement ratio. From the results obtained there appears to be an optimum sand content at a sand/cement ratio of between 0.5 and 1.

As the arm of the investigation is to produce foamcrete products that can be manufactured by rural communities, it is essential to determine how sensitive the results are to small changes in the mix. As the ash and the cement may be obtained in bags, batching may occur per bag, thereby controlling the volumes and densities. The same can not however be said for the sand and although a mix be designed for a specific sand source the moisture content and the exact volume batched may vary.

From Figure 6 it can be seen that a variation in the moisture content of the sand or the sand/cement ratio has a limited effect on the compressive strength of the mix A variation in the foam content does however have a significant influence on the compressive strength and during batching, strict control is required when adding the foam.

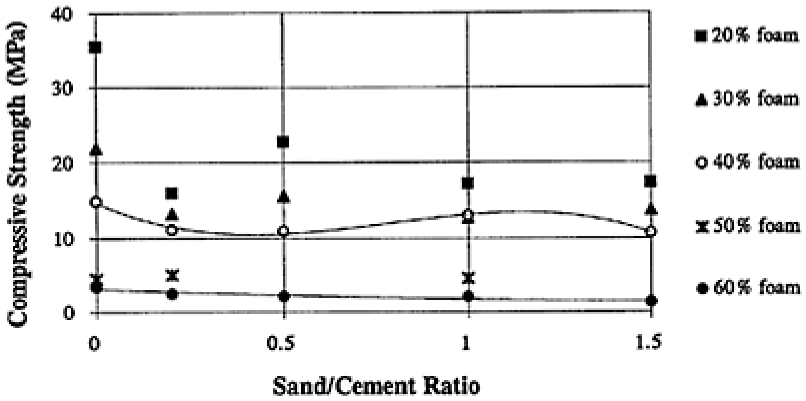

Figure 5: Effect of Sand/Cement Ratio on Compressive Strength

Using an ash/cement ratio of 1 and a sand/cement ratio of 1, two thirds of the cement initially used in the mix has been replaced. Would it now be possible to increase the strength again by adding small quantities of admixtures? We have conducted tests using limited volumes of condensed silica fume (CSF). Silica fume to the extent of 2.5%, 5%, 7.5% and 10% of the mass of the cement was used and the results obtained are indicated in Figure 7. From the results it can be concluded that the silica fume does not have a marked effect on the strength of the mixes with high percentages of foam, but for mixes with low percentages of foam adding 7.5% silica fume results in compressive strengths equal to that obtained before adding any sand.

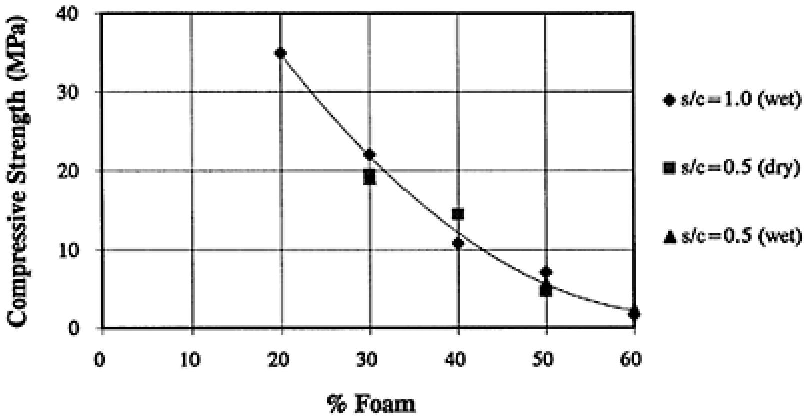

Figure 6: Effect of Foam, Sand and Moisture Content.

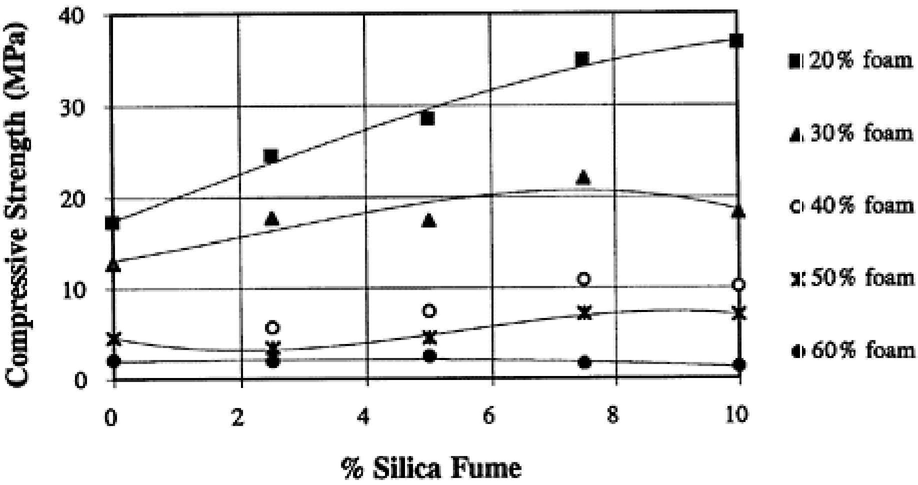

Figure 7: Effect of Silica Fume on Compressive Strength

The compressive strength can be increased even more by adding water reducing agents. The water demand of a mix with a cement:ash: sand: silica fume ratio of 1:1:1:0.075 can be reduced by up to 20% by adding super plasticizers. The results indicate an optimum compressive strength may be obtained for the mixes with 0.75% super plasticizer. These strengths are higher than that obtained with pure cement slurry mixes (ash/cement ratio=0 and sand/cement ratio=0).

From the results obtained to date there seems to be an optimum mix for a specific percentage foam added. To find an explanation for the results obtained SEM (Scanning Electron Microscope) samples have been prepared. Indications are that the foam causes wall effects and the spherical shape of the foam bubbles results in fewer cement particles situated next to the void [5]. When the foam disintegrates, a void is left behind allowing the formation of large calcium hydroxide ($Ca(OH)_2$) crystals. When condensed silica fume (CSF) is added to the mix, the particle size is so small that the packing distribution is not affected by the presence of the foam bubbles. When the $Ca(OH)_2$ crystals forms in the void, the CSF reacts with the $Ca(OH)_2$ to form a much stronger tobermorite gel [4].

Shrinkage

Shrinkage tests conducted by us to date indicate that the shrinkage is directly related to the percentage foam added to a mix. Foamcrete shrinks between 4 and 10 times as much as normal concrete. The shrinkage for a specific foam content can be reduced from that obtained for a simple cement mix by replacing some of the cement with ash and sand and reducing the water content of the mix (by adding a superplasticizer).

Values obtained during shrinkage tests are indicated in Figure 8. These values indicate that the shrinkage is directly related to percentage foam added. For 30% and 60% foam the shrinkage can be significantly reduced by limiting the cement content.

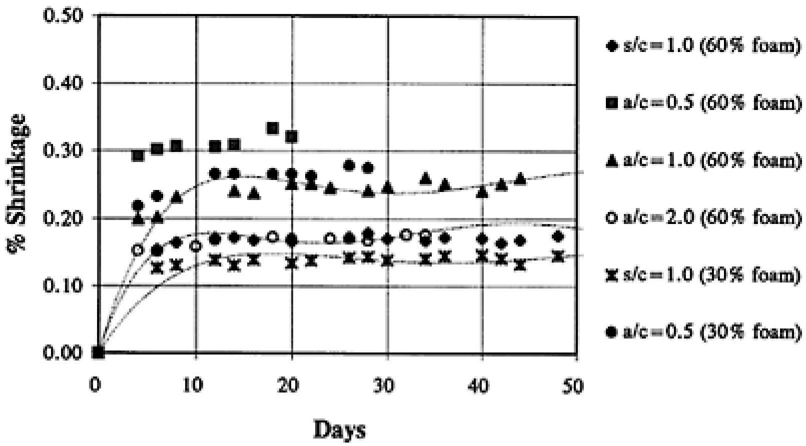

Figure 8: The Effect of Percentage Foam and Mix Design on Shrinkage

PRODUCTS AND USES OF FOAMCRETE

Foamcrete seems to be more suitable to the manufacture of small building elements due to the relatively high shrinkage and low strength of the material. As the material shrinks and expands due to variation in moisture content provision should be made for possible movement or precaution should be taken to prevent changes in moisture content. The most obvious and general use for foamcrete is for building blocks. Tests have been conducted and based on initial results walls built using foamcrete blocks act similarly to those built using ordinary bricks. As blockwork depends on weight when resisting horizontal forces, care should be taken that blocks are not too light to resist forces. The loss in weight as a result of drying can be up to 25% of the initial weight, and this lower density has to be taken into account in design.

Foamcrete does not have a high resistance to abrasion. When using foamcrete in an abrasive environment it is recommended that composite construction is considered, where a strong outer layer protects the inner layer. This method could prevent moisture absorption by the foamcrete and therefore prevent swelling and shrinking. Foamcrete lends itself to precision moulding. As foamcrete has a high percentage of air entrainment, it tends to be extremely workable. Where alterations in shape needs to be made after casting, the material can be sawn and nailed.

A small low cost housing unit, using tilt-up construction, has been developed at the University. Initial indications are that the material cost for the unit is comparable to that of corrugated iron. The construction cost should be similar to corrugated iron but the thermal advantages of the foamcrete could have far reaching implications for the housing market. The main advantage of this method is that no highly skilled carpenters, bricklayers and builders are required as the walls are cast on the floor and a normal straight edge can be used to ensure a neat surface.

Tests have been conducted on blocks to be used as fire walls. The results obtained from these tests were encouraging as after two hours in a fire burning at more than 500°C and being hosed down, only minor hairline cracks on the surface of the blocks were visible. Experiments are currently underway for developing roof tiles, manhole rings, drainage channels, meter boxes, coolrooms, wall units and building blocks. All these products could be used in the Reconstruction and Development Program currently underway in South Africa.

The next phase of the research will include creep and durability tests as well as tests on the use of reinforcing and the bond thereof. The limited tests conducted to date indicate that durable precast foamcrete elements, manufactured by rural communities, could be designed for use not only in houses and infrastructure but as structural members in double storey schools and clinics.

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SOME ASPECTS OF THE DESIGN AND PRODUCTION OF FOAMED CONCRETE

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ABSTRACT. Foamed concrete is finding increasing use in a number of applications, including road reinstatement, void filling and ground treatment. The materials consists of sand-cement mortar into which large amounts of air have been entrained. By adjusting the amount of air in the mix the density, strength and insulation characteristics of the material can be adjusted over a wide range to meet the specific requirements. The reported work comes from an extensive laboratory study to optimise the design and production of foamed concrete mixes. Over a hundred trial mixes were produced, encompassing different sources of cement and sand, varying water-cement and sandcement ratios, and different foaming systems and admixtures. A number of projects were supplied with foamed concrete in parallel with the laboratory study. From the work undertaken it is apparent that the strength at any foamed concrete density can be maximised by selecting a fine aggregate with an appropriate grading and using a base mortar mix with a relatively high workability and water content. Protein based foaming admixture appears to give rise to greater strengths than synthetic based foaming admixture, although the use of a synthetic admixture through a water pressure driven generator proved to be a simple and reliable combination.

Keywords: Foamed, Mortar, Workability, Strength, Density, Grading, Admixture, Ordinary Portland cement (OPC), Ground granulated blast furnace slag (GGBS).

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INTRODUCTION

Foam concrete is a relatively light-weight, low strength material formed by entraining air in a sand-cement mortar. The air is entrained by the addition of either chemical admixture or preformed foam to the base mortar mix.

In situ foam concrete is used in a number of applications which exploit its high workability, low density, low strength and high thermal insulation characteristics. The design of the base mortar and the entrained air can be adjusted to give plastic densities in a typical range of 400kg/m^3 to 1600kg/m^3, 28-day compressive strength values between 1N/mm^2 and 10N/mm^2, and thermal conductivity values between 0.1 and 0.7 W/m°C [1]. The main uses of the material are in road reinstatement and underground void filling. Other applications include as a fill material in ground engineering and as a lightweight, insulating screed [2].

Until relatively recently the production of foam concrete in the UK was very limited, with Pioneer Concrete (UK) Ltd being one of only three major concrete suppliers to offer the product [3]. However, changes to the building regulations in 1990 and the specification for road reinstatement in 1991 have led to an increased interest in the material to speed road reinstatement and enhance the thermal insulation of new buildings [4,5]. Other recent uses of the material in the UK include emergency ground stabilisation after the Heathrow Express tunnel collapse and as a lightweight backfill to create an approach embankment to a bridge [6].

This paper summarises an extensive study of the design and production of foamed concrete undertaken by the author in 1989 and 1990 for Pioneer Concrete (UK) Ltd. The paper concentrates on the laboratory work, although some practical applications of foamed concrete are briefly described.

SCOPE AND OBJECTIVES

The objective of the study was to examine current methods of design, production and utilisation of foamed concrete in order to increase the commercial potential of the material. Within this broad objective it was hoped to give guidance on the optimising the base mortar mix and the addition of the foam element in order to improve the strength and cost effectiveness of the material.

A large number of factors were encompassed within the laboratory work, including: 3 foaming systems, 5 foaming admixtures, 7 fine aggregate sources, and 6 aggregate-cement (a/c) ratios.

In addition to the above, two cementitious types and curing regimes, as well as a large number of plastic densities and water-cement (w/c) ratios, were investigated. The density and compressive strength of the foam concrete were the key properties measured. Limited data on the workability and air content of the fresh concrete were also recorded, although these data are not reported in detail here.

LABORATORY MIXES

MATERIALS

Portland cement complying with BS12 and ggbs complying with BS6699 were used. The main sources of fine aggregate used were a natural sand from Hogshead Quarry, having a BS882 limits F grading, and a 50/50 blend of this sand and limestone fines from Aberduna Quarry, having a BS882 limits M grading.

The main foaming admixtures used were F292 and F294 from Fosroc. Limited work was also carried out with Hex and Regular Protein from Chubb, and Poro BG from Sika. All these agent are synthetic surfactants, with the exception of Regular Protein from Chubb which is based on natural proteins [7]. These agents were used in air and water pressure driven foaming systems as recommended by the manufacturer and described below. There was also a limited use of an accelerating admixture, Conplast NC, from Fosroc.

MIXES

The constituents for the base mortar were batched to within ±0.5% and mixed for five minutes in a 20litre capacity horizontal axis drum mixer. The densities of the base mortar mix and foam were measured using calibrated containers and from these values the mass of foam (Fm) to be added to the mix was calculated as,

$$Fm = Bm \times F_d \left(\frac{1}{T_d} - \frac{1}{B_d}\right) \qquad \textbf{(1)}$$

where, B_m=base mix mass, F_d=foam density, T_d=target density, B_d=base density.

Freshly produced foam was folded in to the base mortar mix using the theoretical dose as a guide until the approximate target density was attained. Following fresh concrete tests, two 100mm cubes were cast from each mix for testing at 7 and 28 days. These cubes were cured in tap water at 20°C in accordance with BS1881, Part 111. Where compressive strength values of less than 2N/mm^2 were anticipated a triaxial loading rig with automatic loading and a 20kN proving ring was used instead of a standard compression testing machine.

The proportions of the mixes produced have been summarised in terms of the a/c and w/c ratios. The latter is the nominal free w/c ratio, allowing for surface moisture on the aggregate, but excluding any moisture in the added foam.

DISCUSSION

Foaming Systems and Admixtures

Foaming system from Fosroc, Sika and Chubb were used in conjunction with their corresponding foaming admixtures.

In the Fosroc system a 4% solution of the admixture is prepared in a large vessel and forced by an electric pump through a hose to a nozzle. The jet formed impinges on a fine gauze forming a stable white foam at a density of 25kg/m^3 and rate of approximately 15 1/s. The Sika system differs from the above in that the solution is pumped using a compressed air driven suction pump to a lance. Here it is mixed with compressed are and blown through a bed of beads to form a stiff white foam. The foam, which resembles shaving foam, has a density of approximately 20kg/m^3 and is discharged at rate of approximately 7 1/s.

In the Chubb foam generator, water under mains pressure passes through a venturi-flume inductor which draws admixture into the flow from a small canister. The orifice can be altered to give either a 3% or 6% solution. The solution passes though a short length of hose to a nozzle. The jet produced passes through a relatively open wire mesh. The foam formed depends on the admixture used. In the case of the protein-based admixture the foam has a large bubble size which breaks down rapidly and has a variable density between 30kg/m^3 and 100kg/m^3.

Problems were encountered early in the work when using Fosroc F292 admixture. At temperatures below approximately 10°C the admixture was found to go milky in appearance and produce foam with an unexpectedly high density (over 70 kg/m^3). This problem eventually led to the replacement of F292 by Fosroc F294, which is formulated for use in cooler climates.

Table 1 shows 28-day strength values for a foamed concretes produced using the different admixtures, from a base mortar mix with a nominal a/c ratio of 4 and w/c ratio of 0.85. The order of strength performance would appear to be Regular Protein >Poro BG≥F292>Hex. This increased strength associated with protein based admixture is consistent with previous research [4].

Table 1 Effect of foaming admixture

FOAMING ADMIXTURE AND DOSE	PLASTIC DENSITY kg/m^3	COMPRESSIVE STRENGTH N/mm^2	
		7-day	28-day
Regular Protein 3%	1342	3.4	4.2
Regular Protein 6%	1350	3.2	3.8
Hex 6%	1374	2.2	2.7
F292[1] 4%	1350	2.2	3.1
Poro BG[1] 4%	1350	2.3	3.3

[1] from strength-density relationship

Effect of A/C Ratio

Figure 1 shows the change in compressive strength with plastic density at different a/c ratios for a base mix containing Hogshead/Aberduna blended fine aggregate. The base mixes were batched to have equal workability, assessed visually. The foamed concrete produced using F292 had a typical flow of 450mm.

It is apparent that compressive strength increases non-linearly with increasing plastic density. A logarithmic transformation of compressive strength was found to produce an approximately linear plot over a wide interval of plastic densities, consistent with an exponential relationship between strength and density.

The compressive strength at any density increases with decreasing a/c ratio, although the effect becomes less apparent at lower densities and compressive strength values. However, as indicated in the plot of cement content and plastic density, as the a/c ratio decreases the cement content increases. The cost of a foamed concrete mix is largely determined by its cement content followed by the foam dose. When strength is the only criteria, optimising the mix design to achieve the lowest cost tends to favour higher a/c ratio and density mixes with low foam doses. Taking the example of a mix to be used in reinstating category 1 road and having a mean 28-day compressive strength of 5 N/mm^2 [4]. From Figure 1, the plastic density required for an a/c ratio of is 1400 kg/m^3, giving a cement content in the foamed concrete of approximately 400 kg/m^3. The corresponding density for an a/c ratio of 5 is 1550kg/m^3, giving a cement content of 240 kg/m^3. For applications such as lightweight insulating screed, the need to minimise the density may overrule cost and favour lower a/c ratio mixes and higher foam doses.

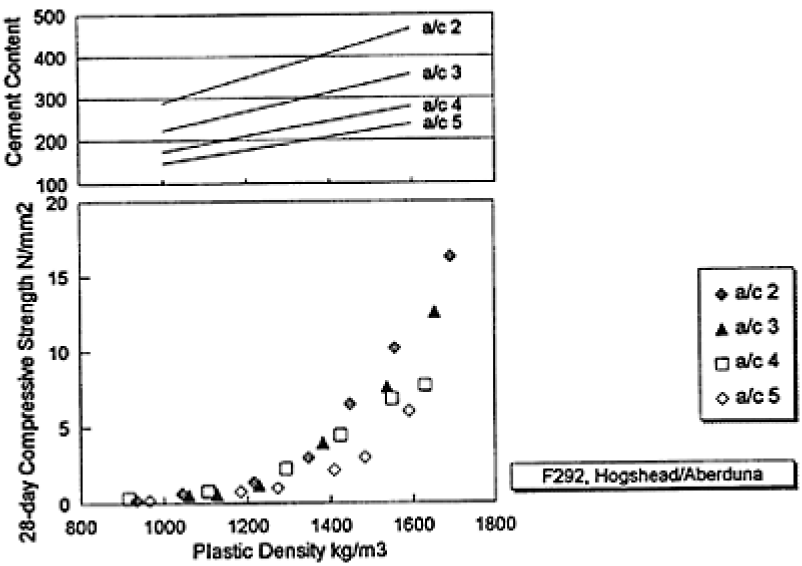

Figure 1 Effect of A/C ratio

Effect of W/C ratio

Figure 2 shows the relationship between the 28-day compressive strength and plastic density for nominal w/c ratios between 0.65 and 1.00. A linear relationship has been assumed between compressive strength and plastic density as a reasonable approximation for the limited range of plastic densities encompassed.

For normal concrete the compressive strength typically shows a non-linear reduction with increasing w/c ratio. In contrast, it can be observed that for these foamed concrete mixes the compressive strength tends to increase with w/c ratio. This trend, which is consistent with work by others [4], is clearly shown by the plot of strength and w/c ratio, inset in Figure 2. This plot shows that at a plastic density of 1350 kg/m^3 there is a progressive linear increase in compressive strength from 1.7 N/mm^2 to 3.5 N/mm as the w/c ratio increases from 0.75 to 0.9.

The explanation for the effect of w/c ratio upon strength was considered to be outside the scope of the study. However, the results suggest that increasing the water content in the foamed concrete has a beneficial influence upon its structure. At w/c ratios above 0.9 and below 0.75 the compressive strength appears to remain static, suggesting that other strength determining mechanisms are becoming predominant. Research by others indicates that the dispersal, stability and shape of the air bubbles is an important factor [8]. Taylor advises that ‘It is possible that more workable or less harsh mixes containing finer materials ensure a more uniform distribution of the entrained air bubbles which results in a stronger foamed concrete’[4]. Regardless of the explanation it is clear that in order to maximise the strength of the foamed concrete the workability of the base mortar mix should be relatively high and the w/c ratio needs to be very carefully selected.

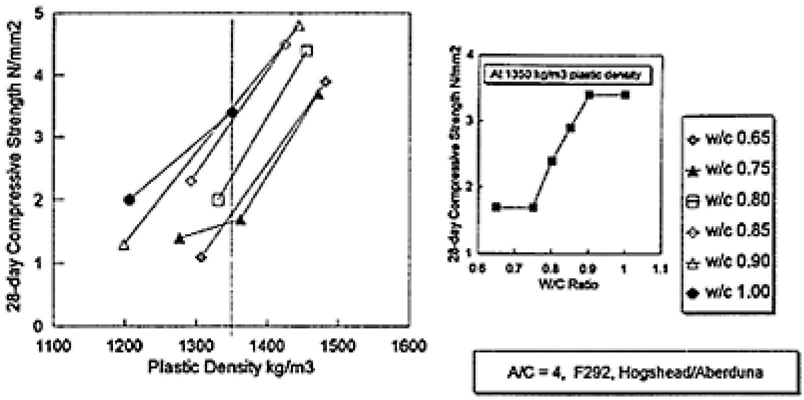

Figure 2 Effect of W/C ratio

Effect of Aggregate Source

The influence of a number of aggregate sources upon the relationship between compressive strength (on a logarithmic scale) and plastic density is shown in Figure 3. All the mixes were produced to the same a/c ratio of 2 and w/c ratio of 0.64. The sands used vary significantly in fineness, with Waddington Fell Coarse meeting limits C of BS882, and Lytham, St Annes sand being finer than limits F of BS882.

If fineness is summarised in terms of the percentage passing a 600 micron sieve, then there is a general increase in strength with fineness. For example, at a plastic density of 1000 kg/m^3 Waddington Fell Coarse with a percentage passing of 53% yields a strength of approximately 0.5N/mm^2, whereas Hogshead sand with a percentage passing of 96% produces a strength of over 2N/mm^2.

This trend, which appears to be maintained up to a plastic density of around 1400 kg/m^3, is consistent with the findings of others [4]. At plastic densities above 1400 kg/m^3 coarser graded sands appear to yield strength values equivalent or higher than those produced by the finer sands.

Whilst the use of percentage passing the 600 micron sieve is a convenient and commonly used parameter for judging the suitability of sand, it is clear from the results for Newark and Waddington Fine sand that this approach has limitations.

Taylor has reported grading envelopes for sand to be used in foam concrete production [4]. These envelopes indicate that the preferred sand would have a percentage passing the 300 and 600 micron sieves of 35–80% and 50–100% respectively. It is interesting to note that all the sands used in this study meet this criteria, with the exception of the Lytham sand which, at the same time, gave rise to the highest strength values.

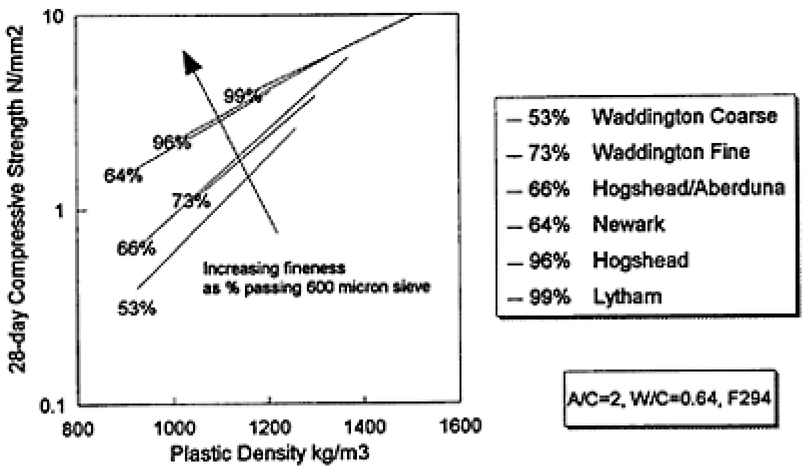

Figure 3 Effect of aggregate source

Effect of Chemical and Mineral Admixtures

The laboratory study included the limited use of a chemical admixture, Conplast NC accelerator, and a 'mineral admixture', ggbs. Figure 4 shows the relationship between compressive strength and plastic density for foamed mixes, with and without the admixtures. The mixes for studying the effects of ggbs and accelerator have a/c ratios of 4 and 1.7 and w/c ratios of 0.72 and 0.42, respectively.

It can be observed that at 7 days the compressive strength of the mixes containing 50% ggbs is significantly below that of the 0% ggbs control mixes. For a plastic density of 1450 kg/m^3 the strength of the 50% ggbs mix is only 1.5N/mm^2 whilst that of the 0% ggbs mix is approximately 3N/mm^2. In contrast, at 28 days the strength position is reversed. For a plastic density of approximately 1450 kg/m^3 the strength of the 50% ggbs mix is 3.8 N/mm^2, whilst that of the control mix is approximately 3.5N/mm^2.

The effect upon 7-day compressive strength of using Conplast NC at the manufacturer's recommended dosage rate is shown in Figure 4. It can be observed that any beneficial effect upon strength of using the accelerator is marginal. The 28-day compressive strength results also support this finding.

The above results, taken in combination, suggest that modification of the early strength is best achieved using different mineral admixtures or cementitious types rather using chemical admixtures. Any admixtures, regardless of type, should of course be checked for compatibility with the foaming system and agent before use.

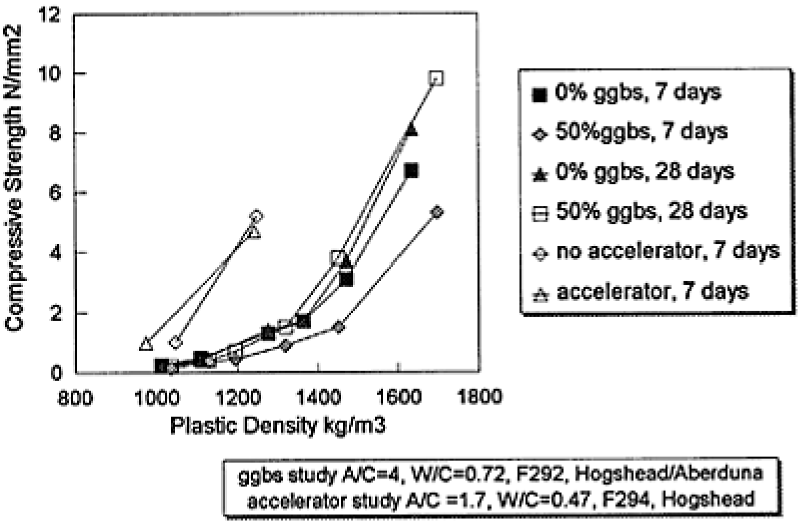

Figure 4 Effect of admixtures

PRODUCTION MIXES

GENERAL METHODOLOGY

The Fosroc foaming system and foaming admixture F294 were used throughout the work. The Fosroc foaming system required a 110V supply. This was provided by a portable generator which was also used to power electronic scales used for density measurements.

A limits M natural sand was used and the base mortar mix was batched to a high workability, consistent with the laboratory work.

The foam dose was calculated as volume based on the target and base mortar mix densities in a similar fashion to Equation 1 and then converted into a time using a typical discharge rate in litres per second for the system. The foam addition typically took approximately 5 minutes, followed by 5 minutes of mixing to thoroughly entrain the foam.

GAS PIPE ENCASEMENT

The project involved encasing a new gas main in foamed concrete reinforced with steel fabric. A total of $135m^3$ of foamed concrete was supplied. Much of the pipe was laid within the highway using a relatively narrow trench. Concrete was simply placed into the trench using the mixer truck chute. In order to reduce nuisance spillages onto the highway, the client provided a purpose made, funnel-shaped extension to the chute.

A portion of the pipe was laid across a school field and this necessitated pumping the concrete horizontally for over 30m. A very small capacity Sydewinder pump was successfully able to pump the concrete over the distance required. Due to its flowing nature it was possible to move large volumes of the foam concrete along the trench using a spade, thereby reducing the distance for pumping.

The details of the mix are as follows:

a) a/c ratio of 3.0 and w/c ratio of 0.65
b) target density and 28-day compressive strength 1600 kg/m^3 and 8–9N/mm^2
c) actual density 1647 kg/m^3 (standard deviation 85 kg/m^3)
d) actual 28-day compressive strength 9.1 N/mm^2 (standard deviation 2.2 N/mm^2)

CELLAR FILLING

Approximately $16m^3$ of foamed concrete was delivered to fill a redundant cellar. The concrete was discharged into the cellar through a ground level access cover. Foamed concrete was chosen because of the low load it would impose on the foundations and walls. The details of the mix are as follows:

a) a/c ratio of 4.0 and w/c ratio of 0.75
b) target density 1400 kg/m^3 and minimum 28-day strength 2 N/mm^2
c) actual density 1395 kg/m^3 and 28-day strength 5.4 N/mm^2 (min. 3.5 N/mm^2)

CONCLUSIONS

Laboratory studies have shown that the compressive strength at any density can be optimised by careful selection of the constituent materials and the base mix proportions.

The water pressure driven foaming system and agents from Fosroc, which were used for the majority of the laboratory work and in the production mixes, proved reliable. However, for cooler climates, where ambient temperatures less than 10°C may be encountered, F294 should be used instead of F292. Protein based foaming admixture yielded slightly higher strength values than synthetic agents.

Higher compressive strength values can generally be obtained using a finely graded aggregate or relatively high w/c ratio in the base mix. Where strength and cost are the only criteria, mixes with high a/c ratios and plastic densities will be favoured. The early strength was found to be influenced more by the use of a mineral admixture (ggbs) than a chemical admixture (accelerator).

ACKNOWLEDGEMENTS

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BUILDING UNDERGROUND—THE CONCRETE CONTRIBUTION

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Appropriate Concrete Technology. Edited by R K Dhir and M J McCarthy. Published in 1996 by E & FN Spon, 2–6 Boundary Row, London SE1 8HN, UK. ISBN 0 419 21470 4.

ABSTRACT. The paper provides a brief review of the role played by concrete in underground construction.

Keywords: Tunnelling, Precast, Segmental Lining, Slipform, Immersed Tunnels, Pipejacking.

Mr Martin A Clarke is Director of Marketing with the British Cement Association, Crowthorne, Berkshire. He is Chairman of the Reinforced Concrete Council, the Ready-mixed Concrete Bureau, the Structural Precast User Group and the Traditional Housing Bureau. He represents BCA on the Council of BRITPAVE and the European Centre Building Project Implementation Committee and serves on many other industry committees. He is Editor of Concrete Quarterly.

INTRODUCTION

This paper briefly reviews developments in the role of concrete underground, drawing on recent examples. There is renewed interest throughout the world in the possibilities of building underground roads, utilities, retail and office developments the author hopes at the conference to stimulate interest in a new forum and database—a British ‘Underground Space Centre’.

Historical Precedent

There is very little that has not been tried in civil engineering “under the soil” in the past hundred years.

The means and methodology of construction over this timescale, of course, has changed dramatically. Take the dawn of the modern tunnelling era, the Thames Tunnel built by Marc Brunel. It took hundreds of men, horses, winches and hand pumps working night and day moving at a rate of 7ft a week, it took five years to dig out half the tunnel length of 600ft. It took a further 10 years to finish it, in between two major breaches of the tunnel when the Thames poured into it!

What was the point of it all? The tunnel was needed to reduce the journey time and alleviate congestion of wheeled traffic over London Bridge and also to reduce the number of ferry boats milling on the river blocking the progress of cargo clippers and sailing barges. Around 1820 some 4,000 slow moving, horse driven carts and wagons crossed London Bridge daily whilst 350 Thames watermen ferried 3,700 people across the river nearby. Some congestion and some unmentionable pollution to go with it. Why was there such a build up of traffic?

Rotherhithe on the south side of the Thames was the industrial hub of London, flourishing with industry and enterprise, packed with mills and wharves, factories and working people. Unfortunately the heart of London's docklands was situated at St Katherine's, and at the London and the West India Docks on the north side of the Thames. The docks took in vast quantities of foreign goods, some of which had to make the four mile journey by road via London Bridge to get to Rotherhithe, to the factories and warehouses. The bridge was not wide enough to cope with extra traffic. Building a new bridge would further disrupt the river traffic and exacerbate road congestion for quite some time. Brunel's tunnel idea was an inspiration.

However, the economic failure of Brunel's tunnel, the horrendous time it took to complete, the catastrophes during construction and its relatively short life thereafter, had a sobering effect on London's enthusiasm for digging under the Thames. It was not until the year 1870 that another passage under the Thames was attempted, when the illustrious James Greathead, a South African born civil engineer, offered to excavate a tunnel under the Thames from Tower Hill to Vine Lane for The East London Railway Company for £16,000 in less than a year! A lot less time than Brunel's attempt which took 15 years and £614,000. The Greathead shield weighed only 2 tonnes compared to the Brunel's shield which weighed 120 tonnes. It advanced at four and a half feet a day. The tunnel linings were eighteen inch thick cast iron segments, which were grout packed tight using hand syringes. The Greathead shield was the forerunner of the modern tunnelling machines.

Developments in subterranean civil engineering today are a worldwide phenomena. Beneath the skylines of London, Los Angeles, Lisbon, Lille, Leningrad, Lesotho and Lausanne lies an underworld city of buried rivers and sewers, railways, pipes and passages, tubes and tunnels. They weave and wind their way to run their urgent errands carrying people, delivering water, removing sewage, conducting power, piping gas, sending messages and conveying parcels. They are the arteries of the modern city, an essential network that imperceptibly controls and maintains the equilibrium and continuity of an ever-changing and volatile world above ground.

Trenchless technology, cut and cover excavation, the slurry diaphragm wall, the immersed caisson, augured piling and thrust-bore are just a few of the many methods available today to create holes in the ground as big as a football pitch or as long as the M25. Once formed, these holes and caverns have a myriad of uses, but they all have to be supported in perpetuity, absorbing the external pressures and strains, the differential subsoil movement, resisting degradation from mineral salts and preventing water ingress. The materials for construction have evolved to respond to the demand for ease of assembly, local availability, economic cost and the variable engineering properties, eg fast setting, retarded set, high early strength, mobility, high durability and toughness. Concrete in all its versatile forms has become the material of choice in subterranean civil engineering today.

The world market in trenchless technology in 1992 was conservatively estimated at $3,650 billion and that excludes the large diameter tunnel market, anything above 2.5m diameter. For every tunnel contract using tunnel boring machines, it is reasonable to assume that 25% of the contract sum is the actual cost of the concrete used. The Jubilee Line Extension for instance has a budget of £3.3 billion over three years. The value to the concrete industry of this project is around £800 million. On the River Medway immersed tunnel contract now under construction, over 65% of the total contract sum will be spent in precasting just the 120m long, 28,000 tonne concrete tunnel units.

Some market, some concrete.

The World of Tunnelling

Probably nowhere has the evolution in going underground been more self-evident than in achievement of tunnelling in the past two decades. Not only has the machinery for excavation become very sophisticated, laser-guided, robot-controlled—a modern tunnel boring machine can excavate a 6m diameter hole at the rate of 100 metres per week with a team of 10 men—but the materials used have to match the speed of operations to make the work safe. In the pioneering days the tunnels were lined with bricks and mortar. For the Box Tunnel designed by Isambard Brunel 5,000 bricks were used every day for advancing the tunnel. They were made locally by a team of 100 men working flat out. An inventory of material used in a week included 1 tonne of gunpowder, 1 tonne of candles, 30,000 bricks carted to site by 100 horses to supply a workforce of 4,000 men.

By the end of the Victoria era cast iron segmental tunnel linings were universally the choice for construction. The system was fast, accurate, relatively lightweight and easier to assemble. Greathead used it for the Tower Tunnel. It has been used extensively to form tunnels for the majority of the London Underground network.

In the 60s came the ubiquitous spun concrete pipe, which once established in the pipeline market encouraged the development of large diameter vertically cast pipes, precast manhole rings, box culverts and, of course, precast segmental linings to compete with steel graphite linings in the lucrative transport tunnelling market.

The concrete optim became attractively competitive. The proliferation and demand for environmentally friendly traffic routing systems through the city and the exponential demand for better transport networks has brought a new challenge for cost effective tunnelling methods, new tunnelling machinery and new construction ideas. The mobility and east of handling liquid concrete has encouraged developments in sprayed concrete linings, cast in place tunnel lining and slipform tunnelling methods, alongside the traditional precast segmental lining.

Options on construction methods today gives a choice of tunnelling or boring method to suit the character of the subsoil conditions, the diameter of the hole, the local topography and constraint around the tunnel heading. The ability of concrete to improvise to suit the site conditions, to provide assured material strength and durability, has made it an essential part in all forms of tunnelling work.

In response to market demand, precast tunnel linings have developed in the past two decades from standard bolted segments, to tapered and keyed bolted segments, to smooth bore trapezoidal and curved segments with gaskets to flush fittings (no bolts) trapezoidal wedge lock segments. The scope of the projects has been vast, the spend in many instances far exceeds that for a new town development budget and can match the achievement of the North Sea Oil platforms for comparative scale. The payback in new technological development and research investment for such a market has been most worthwhile.

Brighton's Submerged Balancing Tunnel

Queen Victoria's playground, a place good for sea air and sunshine, a town with a lively cosmopolitan atmosphere has in recent years had a poor reputation for clean beaches. In fact it is so bad that lying on the beach in Brighton can seriously damage your health at certain times of the year. The old Victorian sewer system, a combined foul and surface water system which leaks a lot is long past its best. At certain times of the year when there are high storm flows, the system surcharges and some of the effluent finds its way onto the beaches.

"Operation Seaclean" is Southern Water's proposal to clean up the Southern coastline. At the heart of the project is the £20 million Brighton Stormwater Tunnel which runs 5.2km along the length of the beach some 20m to 50m below. The 6m diameter tunnel is in fact a large overflow or balancing tank which is big enough to absorb all the excess flow from the sewerage system during peak flow times. The sewage is held until it can be pumped back into the system to be conveyed to the treatment works, before it outfalls to sea. The tunnel boring machine—a rejigged version of the one used for moling the service tunnels of the Channel Tunnel—excavated out of the 6m diameter tunnel at the rate of 130m per week. The tunnel was lined with precast concrete segments.

The Chunnel

Just a little over 2.6 million cubic metres, or 6.2 million tonnes of concrete, enough to fill Wembley Stadium to the height of the Eiffel Tower, was used in the construction of the Channel Tunnel. Half a million tunnel lining segments weighing up to 8 tonnes each, were manufactured on the Isle of Grain and transported to site over a three and a half year period.

High quality, high strength and fast manufacture were the prime requirements for the production programme. The tunnel linings were built with a design life of 120 years to withstand the saline environment under the channel. Granite aggregate from Glensanda in Scotland were transported to the Isle of Grain, for producing the $60N/mm^2$ concrete of 40,000 cubic metres of concrete were mixed at Shakespeare Cliff batching plant and supplied to the tunnels on rail mounted re-mixer drums of 6 and 8 cubic metre capacity.

The concrete was transported up to 22km away, weaving its way on rail mounted mixer drums. It presented a major challenge to the concrete technologists in designing mixes to satisfy these onerous working conditions. To start with, all the concrete was designed with a superplasticiser to keep the water cement ratio to a minimum, some were over-dosed to retard the initial set if they were transported long distances. The concrete was designed to meet the varying demands of low water content, slow hydration, high workability, high early strength, low heat gain.

The crossover tunnels, a vast cavernous space for allowing trains to cross over to the other track, was excavated using the NATM (New Austrian Tunnelling Method). Once the cavern shape was excavated by backacter, the tunnel face had to be rapidly lined with sprayed concrete, before a cylindrical shutter 5.5m long was positioned. The half cylinder over the tunnel arch was reinforced. Concrete was then placed to fill the void between shutter and the sprayed concrete liner. The gap between form face and sprayed liner was over 1 metre thick and needed 200 cubic metres to fill, taking 15 hours to complete. The cylindrical shutter had to be ready for the next section within 36 hours of casting, so the concrete had to have a guaranteed minimum strength of $10N/mm^2$ within 18 hours of placing. It did not prove to be a problem. What to do with all the excavated spoil? The 5.5 million cubic metres of tunnel debris was used to reclaim thirty hectares from the sea opposite Shakespeare Cliff, thus creating an amenity for uses linked with the Tunnel, So before things got under way, a seawall 1,700m long was built with massive bund walls to stop the sea from breaking through. Two rows of sheet piles spaced 10m apart were driven into the sea bed to form the sides of the wall. The gap was filled with concrete to a height of 12m. It took three years and 210,000 cubic metres of concrete to complete the bund. Self-compacting concrete was transported nearly 1,000m to reach the discharge point below water level. A cohesive concrete suitable for pumping with a high cementitious content was necessary. To keep hydration temperatures under control 40% replacement pfa was incorporated in the mix. To complete this part of the project, a revetment and wave wall apron 9m wide was built along the-length of seawall bund consuming a further 10,000 cubic metres of concrete.

In laying the rail tracks, a pair of independent precast concrete support blocks were placed under the track at 600mm intervals over the entire length of the two running tunnels. Concrete in all its forms, and in all its engineering properties has been used to maximise efficiency of construction. The concrete both shields and protects the myriad of structures required to safeguard the investment in this massive enterprise.

A Ring of Bright Water

Twice as long as the Channel Tunnel and buried 45 metres below the pavements of London and its leafy suburbs runs a 2.5m diameter pipeline carrying 50% of London’s treated water (1,320 million litres per day). This is an underworld of thrust boring, impact moling, rod pushing, fluid jet cutting, pipe jacking and microtunnelling—the world of “no dig” trenchless technology.

If the hole needed for a pressure pipeline or cable route is less that 150mm in diameter then impact moling, thrust boring or rod pushing are the answer. For larger diameters microtunnelling and fluid jetting can be the preferred options. However, if you want to lay a new gravity pipeline then pipe jacking and microtunnelling are the two serious contenders, depending on the diameter.

There is little appreciation in public circles about “no dig” technology outside tunnelling circles, yet so much innovation and development work has taken place in this field in the past 20 years. Concrete again has been the material of choice, to mould, to fabricate, to pump, to transport materials along the narrow pathways down all sizes of man-made holes, deep into the underground.

Thames Water Authority were puzzling over the problem of how to put in additional service capacity onto their existing water system, to cater for the demand forecasts into the next century. The first and obvious solution was to renew and enlarge miles of trunk mains to extend the capacity of its treatment works, then route them over new land areas. This option was too expensive and totally impractical, because a large part of the new route would disrupt city traffic and because there was simply no room left near the surface to build additional pipelines.

The feasible option was to construct a new ring main from Coppers Mill in East London and form the southern link to Ashford, in the west, via Battersea and Surbiton. The northern link would run via Holland Park, Hammersmith and Kempton before connecting to Ashford. The encircling tunnel will be connected to a number of water treatment works in the catchment area through the existing network. The ring main will be used to shunt water from one network to another, to balance the distribution network and rationalise total demand into the next century. Water from the ring main will be pumped directly into the distribution network through 12 strategically located shafts. Each one will be 12m in diameter and will be capable of handling flows sufficient to sustain a population of 200,000.

The project is valued at £260 million and will utilise 280,000 tonnes of concrete when complete. 80% of the concrete is in the form of unreinforced precast concrete wedge block lining—first pioneered in London in the late 50s—and segmental tunnel linings in the distribution shafts.

Immersed Tunnels and Dutch Ingenuity

Underground expertise does indeed exist away from these shores. The UK can boast of two major immersed tunnel projects in the last decade that rank among the biggest in the world. The Conway Crossing completed at the end of 1989 used six 30,000 tonne concrete tunnel units 118m long by 24.1m wide. They were floated out into the Conway and ballasted into position on the Estuary bed to form a four lane highway. The Medway Immersed Tunnel is now virtually complete. It comprises 370 metres of immersed tunnel sections, the longest of which is 126m long a reinforced concrete unit weighing 26,000 tonnes.

Credit for immersed tunnel technology must go to the Americans who were the first to pioneer its use. But the Dutch have made it the exclusive technology of tunnelling in the Netherlands. Surrounded by the sea, a high water table, many wide canals and a subsoil that is mainly sand present problems which make conventional tunnel boring difficult. So instead of digging deep, the tunnel is precast and floated out and then immersed onto the bed of the waterway. This form of construction is tailor made for the Netherlands.

It has been popular for large tunnel projects under canals, rivers and estuaries since 1960. It does require a large site for precasting the tunnel units—really huge box culverts containing both halves of a dual carriageway. The units are built in a dry dock very adjacent to the final position line. The dry dock is then flooded with the units sealed at each end. They are buoyed up with pontoons, then towed out and immersed into place.

The Coen Tunnel running under the Noordzee Canal was built to replace the inadequate traffic capacity of the low level movable bridges connecting the harbours of Amsterdam, Rotterdam, Dordretch and the Moerdijk region with the North Sea. With only lifting bridges at the crossover points, it only takes one slow moving cargo ship to cause a traffic jam a mile long in each direction at peak times.

Six 90m long reinforced concrete tunnel units weighing 20,000 tonnes were cast in a dry dock. Before the units were floated, the bulkheads were sealed with concrete and then fitted with a water ballasting system with stopcocks and pumps for filling and pumping out water. The tunnels were ballasted in such a way that it had a net buoyancy of 100 tonnes when partly flooded and attached to the two large buoyancy pontoons. The units were towed into position by a team of tugs, then the pontoons were flooded and slowly but steadily the massive concrete sections were lowered into place adjacent to the previous unit.

Once in place on the canal bed foundation, the tunnel unit is drawn hard against the adjacent section using hydraulic rams. The gasket of the tunnel units form a watertight seal under hydrostatic pressure, when the new tunnel unit is dewatered. The bulkhead of the connecting units is broken out and the roadway is then constructed for the next 90m section.

Immersed tunnels have been used to bridge canals over roadways, to build roadways across airports, rail and metro tunnels under rivers and waterways. Some are 24m wide units, some are 50m wide and contain an eight-lane carriageway for mixed transport use. Probably the longest tunnel section immersed in one piece was the 268m long segment for the Hem Rail Tunnel under Noordzee Canal near Amsterdam.

Other Underground Feats and Uses

In a brief history of civil engineering achievement this century, the 53.9km Siekan Rail Tunnel ranks as the longest and most expensive tunnel ever bored in the world. The Channel Tunnel is the second longest at 51.8km! The 27km £500 million Giant Collision Tunnel in Switzerland, is the largest scientific instrument in the world. Whilst the V2 rocket factory under the Harz Mountains in West Germany was the world’s largest underground factory.

Today underground structures are being used for nuclear waste repositories, sewage treatment works, oil storage reservoirs, water storage, power generation, and not least mineral extraction which is a subject all of its own, with its own engineering characteristics and notable achievements. There just isn’t room enough in this overview to cover the subject fully.

New projects are being tunnelled somewhere in the world every day, whether it is using tunnelling machine, immersed tube, NATM, or pipejacking and concrete whether sprayed, cast in place, slipformed or precast will be doing the job. Buried it may be, but not forgotten.

The Future

The BCA is keen to encourage the formation of a new group to research and promote the development and growth of underground construction of all types in the United Kingdom. Any interested parties should contact the Author.

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Theme 4
BINDER TECHNOLOGY

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Leader Paper

Hydraulic Cements—New Types and Raw Materials and Radically New Manufacturing Methods

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HYDRAULIC CEMENTS—NEW TYPES AND RAW MATERIALS AND RADICALLY NEW MANUFACTURING METHODS

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ABSTRACT. The concept of Sustainable Development in cement manufacturing is introduced. Several alternative ways of energy conservation at conventional, very high and very low temperature technological methods of cement manufacturing are assessed. The most likely effects of these production techniques on the environment and on the reserves of non-renewable fuels are evaluated. A method of cement recycling, or cement rejuvenation is also discussed.

Keywords: Sustainable development, Energy reserves, Energy conservation, Industrial by-products, Waste management, High temperature kilns, Chemical activation, Mechano-chemical activation, Recycling, Rejuvenated cement, Kiln emissions, Greenhouse gases, Environmental effects.

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INTRODUCTION

The future is always hard to predict. Most of the laws of a social, environmental and technological change are based on the principles which are expressed by the equations of non-linear dynamics, or by so called “Chaos” theory [1–3].

It is apparent from the mathematics of this theory, that a very minor change in the initial state of a non-linear dynamic system can lead to the enormous changes in a state of this system, even over a relatively short period of time. For example, the factors influencing weather patterns are the components of a classical non-linear dynamic system, and, as we know, the reliable weather forecasts seldom exceed three or four days.

There are nonetheless several factors influencing the future of humanity which undeniably exist, which are detectable and even measurable with some degree of accuracy [4,5]. These include:

1. <u>The continuous growth of the world's population,</u> which, from a current estimate of just under six billion, is increasing by an average of 245,000 people daily (there are an average of 386,000 birth and 141,000 death every day in the world). The estimates of the earth's population by the year 2050 range from eight to twelve billion [6].
2. <u>The limited area of arable land.</u> The total land area is 135,973,730 square km, and thus the average current population density is 44 persons per sq. km. The important problem for sustainable land development, which is related to the future food supply, is to some extent the land surface can be more completely utilised, while supporting an increasingly higher population. It is estimated that at present about one-third of the earth's total land area is capable of being cropped, but the main limiting factor remains the lack of water. In the early 1970-s only about one-fifth was so used, of which approximately ten per-cent was irrigated. It should be noted, that in many parts of the world continuous irrigation leads to the significant increase in soil salinity, and hence to a potential loss of arable land.
3. <u>The limited reserves of fossil fuels.</u> Total reserves of fossil fuels in the world are currently estimated at approximately 11.3 teratonnes (i.e. 11.3×10^{12} tonnes) of coal equivalent, of which (at the current consumption rate) coal reserves of 8.5 teratonnes should last approximately 220 years, and the oil reserves of 0.5 teratonnes could last about 30 years. Possible discoveries of new reserves may extend this estimate by another 10, or 20 years at the most.

   There are additional reserves of gas of about 0.3 teratonnes and of the fuels extracted from oil shales and tar sands, which may be as high as 2.0 teratonnes of coal equivalent. These may replace oil, when it runs out. However, if the demand on energy from fossil fuels will increase proportionally to the growth of population, the reserves of all fossil fuels will last significantly less than 200 years,—a very likely scenario, judging by the current trends, and,
4. <u>The inevitable increase in the environmental pollution, resulting from an uncontrolled population growth.</u> Specific to cement industries, the main effects on water pollution are: high probability in the increase of pH, and the potential danger to aquatic life. However, in some cases an increase in pH of water may actually be beneficial, as it may neutralise the harmful effects of acid rain. Other factors include discharge of warm water, resulting in the reduction of dissolved oxygen in rivers and lakes and in the abnormally high rate of plant growth or eutrophication (e.g. blue-green algae).

   The most likely effects on air pollution are: increase in greenhouse gases, particularly in the carbon dioxide, as well as increase in the concentrations of oxides of sulphur, nitrogen, etc., resulting in an acid rain, photochemical smog, etc..

As the growth of these factors approaches their limiting value, the catastrophical failure of the system which relies in its performance on these factors, becomes inevitable.

Thus, the system must adapt to the change, in order to survive.

Manufacturing of hydraulic cements is influenced by all four of the above factors. The growth of population represents an increased demand on building and construction materials, and thus on the output of cement; there is already a fragile balance in many parts of the world between the land allocated to forests, farming and quarrying; the use of fossil fuels in cement industries accelerates the depletion of these non-renewable energy resources and contributes to the build up of “greenhouse” gases in the atmosphere.

Therefore cement manufacturing industry, as any other, must implement a method of sustainable development, in order to achieve at least some degree of control of the factors, mentioned above.

Sustainability is the ability to maintain a desired condition over time [7,8].

Sustainable development *is a tool for achieving sustainability,* not a desired goal.

SUSTAINABLE DEVELOPMENT IN CEMENT MANUFACTURING

Overview of the Methodology

The sustainability in cement manufacturing must, therefore consist of the following elements:

I. Energy conservation in the conventional technologies of cement manufacturing, viz,—(a) use of wastes as fuels, (b) use of fluxes to reduce temperatures for clinker formation, (c) use of raw materials requiring lower temperatures for clinker formation, and (d) utilisation of industrial by-products and wastes as potential raw materials in cement manufacturing.

These changes can be implemented in the existing plant with little or no modifications necessary and thus, can be most easily introduced in the near future.

II. Energy conservation by the development of new technologies of cement production. This can be done by manufacturing cements:—(a) at conventional kiln temperatures of 1400 °C, but using new types of raw materials and producing cements with new and unconventional properties and behaviour,—an extension of <I (d)>—and by using new methods of clinker formation, such as fluidised bed kilns, (b) by developing new technologies of clinker formation at very high temperatures, so that the rate of reactions of clinker forming products is very fast indeed, and (c) producing cements at very low temperatures, by introducing technologies of chemical and mechano-chemical activation of seemingly inert materials, industrial by-products and wastes.

III. By creating new types of cementitious materials from recycled concrete.

All of the above techniques should be evaluated not just from the point of view of energy cost-effectiveness, but also by assessing the potential effects of kiln exhausts, and the effects on air and water pollution of these new production methods.

New cements, manufactured by the new technologies should also be evaluated for the way in which the end products made from these cements interact with the environment.

Let us consider some of the above technological changes one by one.

ENERGY CONSERVATION IN CONVENTIONAL METHODS OF CEMENT MANUFACTURING

Use of Wastes as Fuels

Formation of ten tonnes of clinker in a rotary kiln requires approximately one tonne of fuel oil. One tonne of fuel oil is capable of generating some 4,000 kilowatt hours of electricity. To make cement, clinker has to be interground with additives. Grinding uses about 40 kilowatt hours per tonne of clinker, or about 10 kg. of oil. Thus, to produce one tonne of Portland cement, 110 kg of fuel oil is required.

Since the middle of 1980-s the use of many combustible solid and liquid wastes has dramatically increased as an extension or as a partial replacement of conventional fossil fuels in cement manufacturing. In the United States by 1991 some 1.3 million tonnes of pumpable wastes were used as a fuel in kilns, replacing one million tonnes of fossil fuels.

One typical example is the use of spent lubricating oil, with a calorific value of around 37 MJ/kg, which is only slightly below the 42 MJ/kg of a fuel oil. Substitution rates of up to 30 per cent resulted in a virtually unchanged composition of the emissions from kilns,—the main restriction which must not be breached when using wastes as a fuel.

Waste motor vehicle tyres is another typical example of a solid fuel substitute. In Europe the general trend is to feed whole tyres into a kiln and the Americans seem to prefer handling systems which utilise shredded tyres.

In Australia, after extensive trials by the Environment Protection Authority (EPA), Blue Circle Southern Cement developed whole tyre handling system at their Waurn Ponds kiln in Victoria in 1992. Some 20% of the kiln fuel requirements are now met by tyre substitute, representing a consumption of around one and a half million tyres annually. The stack emissions were carefully monitored, and the levels of carbon monoxide and particulates were shown to be well within the EPA limits.

Many hazardous waste streams, such as paint residues, chemical process sludges, oil refinery separator sludges, etc., have the potential of being used as fuel extenders in cement manufacturing. Some of these wastes may require pre-treatment, as any residual fuel reaching the sintering zone can cause localised reducing conditions and thus adversely affect the quality of the product.

Relatively non-combustible material like steel in tires or incidental metal from filter cartridges or aerosol spray cans oxidise upon entering the sintering zone which has temperature of the order of 1500°C and oxygen. Iron oxide then exothermally reacts with calcine oxide and ultimately becomes part of the cement minerals. In some cases, if raw feed has an inadequate iron content, the mineralogy and the performance of cement may actually be improved.

The likely adverse change in emission from cement kilns burning waste may result from the higher chlorine content in waste, as compared with fossil fuels. Thus, the measurement of the ability of a cement kiln to destroy chlorinated organic compounds must be a part of every feasibility study of fuel extenders.

However, it seems that cement kilns can destroy even the most stable compounds, such as trichlorobenzene. This can be explained at least in part by the fact, that any chlorine entering the kiln should have several hundred times its equivalent of calcium to react with.

Use of Fluxes and Mineralisers to Reduce Temperature of Clinker Formation

It is well known, that both aluminium oxide and iron oxide act primarily as fluxes in the process of clinker formation. A flux promotes reaction by lowering the temperature of melt formation, and mineralisers accelerate the kinetics of reactions through the modification of a sintering process. The most effective combined flux of aluminium and iron oxides has the weight ratio of A/F equal to 1.38.

There are many other oxides apart from aluminium and iron, which act as fluxes. Oxides of sodium, potassium and magnesium, when used in relatively low concentrations, act mainly as fluxes. It seems that the temperature of formation of melt from a raw meal depends mainly on magnesium and sulphur oxides, and the temperature of formation of melt from clinker liquid depends on magnesium, sulphur and potassium oxides.

Fluoride, sulphate and phosphate act as mineralisers in the Portland cement manufacturing process.

Up to date, the use of fluxes and mineralisers,—if we take the entire process of manufacturing cement into account,—resulted in relatively minor energy savings, when change in the composition and in the properties of Portland cement is very small. However, reductions of the reaction temperatures in excess of 200 °C can be achieved, and offer a potential area of investigation in the production of low-temperature cements of special phase composition.

Use of Raw Materials Requiring Lower Temperature for Clinker Formation

The consumption of energy in production of Portland cement depends on the process (e.g. wet, dry, semi-wet or semi-dry), on the kiln size and type and on the kiln efficiency (e.g. a preheater design). The overall heat consumption in the conventional kilns can vary from 3100 to 6700 kJ per kg of clinker. The improvements can be made using raw materials which consume less energy during the process of firing.

The materials necessary for manufacture of hydraulic binders are essentially oxides of calcium, silica, aluminium, iron and magnesium. Natural raw materials with higher concentrations of oxides, which act as fluxes and/or mineralisers can produce low-energy cements. The properties of these cements would generally differ from those of Portland cement.

At present, only two types of hydraulic cements, to my knowledge, are manufactured industrially in large quantities:—Portland cement, containing between 2 and 4.5 per cent of iron oxide, and aluminous cement with a maximum of 15 per cent of iron oxide.

Aluminous cement, of course, is known to have durability problems in warm and moist environment.

There are other types of iron-rich cements, such as Ferrari cement, in which the raw meal is selected so as to replace most of aluminium oxides with those of iron. This cement has lower energy consumption to fire, by comparison with Portland, and the clinker, although it is harder than that of Portland cement, requires less energy to grind.

Other iron-rich cements are made by replacing, for example, most of silica oxide with iron. This cement is produced from raw material of Portland cement type to which some bauxite is added. The properties are similar to a rapid-hardening Portland cement.

Several experimental cements, one with high proportion of iron ore in the raw meal and another produced by firing limestone with pyrite ash are currently being evaluated for the long term behaviour.

Another interesting new development is the replacement of argillaceous component of the raw meal with basalt. Basalts contain higher percentage of iron and aluminium oxides than clays and shales. The firing temperature can be reduced to 1300°C, and the phase composition can vary over a considerable range, but generally the content of tricalcium aluminate and tetracalcium alumino-ferrite is higher than in Portland cement at the expense of either tricalcium or dicalcium silicates.

Extensive laboratory evaluation so far, indicates good strength characteristics, and reasonable dimensional stability of basalt cements. These results must be confirmed by the long term experimental and field data, including durability tests.

Utilisation of Industrial By-products and Wastes as Raw Feed Components

There are many industrial by-products and wastes which, in the process of their formation, undergo extensive heat treatment. Some of these materials can be used to make hydraulic cements of relatively low energy demand.

Blast-furnace slags, which are produced in ironmaking, consist predominantly of aluminium, silica and calcium oxides, and steel slags, which arise from steel manufacturing processes, are composed mainly of calcium and iron oxides. These slags can be used as main ingredients of raw meals in making hydraulic cements. The properties of these binders are somewhere in between those of Portland and Ferrari cements.

Quite considerable savings of energy can be achieved in the manufacturing process of yet another hydraulic cement, which requires no external energy input. It is produced in two stages. Steel slag at first is treated in the basic oxygen process converter, with the addition of a special compound, consisting of calcium, aluminium, iron and magnesium oxides. During the second stage, the slag, just after tipping, is subjected to an oxygen blow. Its composition is modified during the second stage by adding bauxite or a component of a similar mineral composition. After the treatment slag obtains weak cementitious properties and its use is generally limited to an inclusion in Portland and aluminous cements as an extender.

There are also new cements produced with wastes, which have not been subjected to heat treatment. Calcium silicate based cement is such an example.

One of the methods of manufacturing phosphoric acid involves the reaction of phosphate rock with sulphuric acid. The by-product of this reaction is phosphogypsum. Apart from the main components of calcium and silica, phosphogypsum usually contains traces of phosphorus, iron, aluminium, sodium, potassium and other impurities. A "high early strength" type cement can be produced at 1200 °C, using phosphogypsum. The content of limestone in the raw feed in this cement is lower than in Portland, and the raw meal requires less energy to grind.

A cement similar to Portland can also be produced with higher than conventional content of sulphates in order to obtain the $4CaO3Al_2O_3SO_3$ phase. This cement is also sintered at 1200°C.

Another waste successfully used in the manufacture of hydraulic cements is red mud. Red mud is a by-product from the Bayer process of making alumina from bauxite. It contains oxides of iron (of the order of 40%), aluminium (approximately 20%), silica (some 10%), and then gradually diminishing percentage of calcium, sodium, titanium and traces of some other impurities. The raw meal is composed of limestone (72 to 76 percent), red mud (10 to 13 percent) and amorphous silica, such as lechatelierite (13 to 15 percent). Reductions of sintering temperatures are only of the order to 50°C to 100°C. This cement exhibits good strength and sulphate attack resisting characteristics.

It should be pointed out, that the reduction of firing temperature should not be assessed in isolation from the total energy requirements in manufacturing and use of all these cements [9–16].

ENERGY CONSERVATION BY THE DEVELOPMENT OF NEW TECHNOLOGIES OF CEMENT PRODUCTION

New Raw Materials and Methods of Clinker Formation

Rotary kilns with suspension preheaters and partial or complete precalcining systems represent the current technology of clinker burning. During the past century improvements in the efficiency of clinker making were mostly directed towards the design systems which ensured reductions in the losses of heat from a kiln to the environment.

Partial precalcining gives an increase in a kiln capacity of about 30%. The best performing kilns with complete precalcining have the heat consumption of about 3150 kJ per kg. of clinker. Further reduction of heat consumption can be achieved by using excess heat exit gases for power generation. Another option is a storage of this energy.

The best performing kilns of this type have the unit heat consumption of about 3000 kJ per kg. of clinker. By comparison, some of the old wet process kilns had the unit heat consumption as high as 8500 kJ per kg. of clinker.

The factors affecting a complete physical and chemical process of clinker formation, like any other heterogeneous reactions include, the time of heat transfer, the time of external and internal diffusion and, most importantly, the time of dehydration reactions and the sintering reactions of the clinker forming minerals.

When the rate of diffusion is high and the heat exchange is intensive, then the rate of clinker formation is governed by the chemical reaction kinetics alone. However, the rate of clinker formation will depend mainly on the rates of diffusion and of heat exchange, when these processes are slow.

In order to speed up the process of diffusion and of the mass transfer, it is necessary to increase the flow of furnace gases past the raw meal particles, and to decrease the particle sizes and/or to increase their porosity.

Kiln designers should be considering the combined effects of the rates of firing, temperature, soaking times, kiln atmosphere, etc., as well as the cooling systems. Increase in the rotary kiln efficiency requires modifications to the atmosphere and temperature, or the implementation of completely different firing systems, such as fluidised beds, plasma torches and electromagnetic beams.

These new types of kilns may require non-traditional raw feed, and produce clinkers which differ from Portland both in phase compositions and in properties.

Some of the new cements, which were mentioned above, can be manufactured more cost-effectively in these new types of kilns [9–11,23].

New Technologies of Clinker Formation at Very High Temperatures

When the rate of clinker formation depends only on the chemical reaction kinetics, significant increase in the sintering temperature can lead to an entirely new process of making clinker, and to the new types of kilns.

It is true for the most type of thermal treatments, that when the required temperatures are in excess of 1600°C, the use of electrical energy becomes more cost-efficient than that of fossil fuels.

The use of specially designed magnetohydrodynamic furnaces can ensure the sintering temperatures well above 3000 °C. There are new methods of electric energy generation from biomass, solar, wind and tidal sources of energy, in addition to the traditional hydroelectric and nuclear sources.

The viscosity of a liquid phase of the clinker forming minerals at these temperatures are measurably reduced, and the diffusion coefficients, as well as the rate of chemical reactions are significantly increased.

It is well known, for example, that the eutectic temperature for the $CaO\text{-}2CaOSiO_2$ phase in clinker is of the order of 2065°C.

In spite of such high energy requirements to generate these temperatures, the overall unit energy consumption in these new types of kilns can actually be lower, than in cements made by the conventional manufacturing methods [17, 20, 23–25].

Cement made at Low Temperatures, Using Chemical or Mechano-Chemical Activation Technologies

Industrial binders can be arbitrarily divided into three main groups, according to the nature of their cementing action.

Portland and blended cements (i.e. mixtures of Portland with cement extenders), aluminous and Ferrari cements, plasters, etc., develop hydraulic bonds; Binders containing calcium silicate, magnesium sulphate, aluminium phosphate, as well as some refractory and dental cements develop chemical bonds; Synthetic resins which harden by polymerisation or thermocondensing, develop organic bonds.

The energy requirements in preparation and in the process of hardening of these binders do vary quite significantly, and, depending on the availability of raw materials, properties and applications, the most energy efficient binders should be considered for proper industrial use.

In many cases significant energy savings can be achieved by replacing part of Portland cement with extenders, such as fly ash, rice-husk ash, silica fume, or with similar industrial or natural pozzolans.

Ground granulated blast-furnace slag has very weak hydraulic properties of its own, and is also used as an effective extender of Portland cement. The energy required to grind one tonne of granulated blast-furnace slag can vary from about 40 kWh to 70 kWh per tonne of this by-product. Thus the energy saving is less than that of pozzolanic materials, although properties of the blend may be superior to that of pozzolanic blends in some applications.

The use of blended cements goes back many decades and it is well covered by the world cement standards in all developed countries.

The new developments open up opportunities for using Portland cement or lime, or other alkaline substances purely as activators of cementing properties of slags and pozzolanic materials. This, of course, will represent a major energy saving, in comparison with the use of Portland cement alone. Pozzolans, by definition, have no cementitious properties, unless activated with lime. Activation, however, by no means is limited to lime.

Since the late 1950-s theoretical fundamentals of new types of binders, produced by activating natural siliceous materials, by-products and wastes, were developed in several East European countries.

It was shown, that hydraulic binders can be produced from natural siliceous materials, which are ground to a fine powder, or from industrial siliceous wastes, by activating this materials with caustic alkalis and with salts of alkaline metals to produce so called soilcement.

Sodium metasilicate or water glass and calcium chloride were used to produce new binders under laboratory conditions. It seems that the process of the silicate phase formation in clay minerals mixed with soda occurs in a solid state and progresses steadily from a high-basic (nepheline) to a formation of a low basic (feldspar type) mineral. The process can be accelerated by a heat treatment. The most rapid change from nepheline to albite apparently takes place at temperatures in the range form 900°C to 1000°C.

The strength of these binders is improved by increasing the content of glass phase in the system.

It was also found that the interaction of melilite slags with compounds of alkali metals leads to the formation of cryptocrystalline structure, characterised by very high strength of these binders.

This leads to the development of slag-alkaline cements. By establishing an optimum correlation between the gel and crystal phases, an one day compressive strength of more than 50 MPa was achieved in concretes based on slag-alkaline cements. One month compressive strength was of the order of 160 Mpa. Currently, there is a wide use of slag-alkaline cements in the countries forming CIS (i.e. former USSR).

Another development, which also has an industrial application, is the use of blended industrial wastes (fly ash, silica fume and phosphogypsum) to replace up to 60% of

Portland cement in concrete, which also contains high range water reducing admixtures (surfactants).

A completely different development of new binders has resulted from the application of the fundamental principles of mechano-chemistry. Mechano-chemical activation of seemingly inert siliceous materials resulted in the development of silica-water-suspension-binders or SWSB.

It was shown, that concretes with SWSB, containing alumino-silicates, after hardening and curing at temperatures of between 40°C and 120°C, achieved compressive strength of about 60 MPa. Silica-water-suspension-binders are manufactured in specially designed high intensity planetary mills.

It should be noted that slag-alkaline cement and SWSB based concretes have excellent durability, comparable to that of naturally occurring igneous rocks [18– 22,26].

Rejuvenated Cement

Currently, large amounts of materials from demolished buildings and structures are recycled. This is particularly true for concrete.

It has been shown, that a considerable proportion of cement grains in concrete, which was in service in buildings and structures for very long periods of time, remain only partially hydrated. Coarser grains, with the diameter of greater than 30 microns, and particularly those in excess of 50 microns in diameter, usually contain a core of unhydrated cement in the middle. Lime forms abut 20% of the hydration products in Portland cement.

Thus, fine fraction of recycled concrete can be rejuvenated, using mechano-chemical activation, to become a hydraulic cement in its own right (Patent pending) [14, 22].

CONCLUSIONS

The increasing world wide awareness of the Global problems of overpopulation, energy crisis, and environment pollution facing the current, but particularly future generations, will lead to the establishment of principles of sustainability.

To achieve sustainability in cement manufacturing, we must first of all enforce energy conservation, i.e. replacement of non-renewable fossil fuels with renewable fuels or with wastes.

New types of cements, and new methods of clinker making requiring less energy than the conventional manufacturing methods should be developed. This should be done in conjunction with the study of the effects these new technologies will have on the environment. The study of environmental effects should not be confined to just the dangers of air and water pollution during the clinker making process. It should extend to evaluation of the interaction of materials and products made from these cements with the environment.

The products made with new cements should be energy and cost efficient, environmentally friendly, durable and, in some cases, capable of hazardous wastes encapsulation [27].

There are several new technologies and many new cement types which have been or are being developed, and which could satisfy most of the above requirements. We must put these to good industrial use as soon as practically possible.

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HYDRAULIC BEHAVIOUR OF NON-EXPANSIVE SULFOALUMINATE CEMENT

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ABSTRACT. A sulfoaluminate cement was prepared by sintering a mix of industrial raw materials and by-products at 1280°C. The cement prepared had no expansive properties. The physical and mechanical prorerties of this cement were studied in comparison to a Portland cement. A satisfying behaviour regarding, quick setting, rapid hardening and high early strength development was observed. The heat of hydration was found lower, but the rate of heat liberation in the first hours of hydration was higher than that of Portland cement. The lower porosity of the sulfoaluminate mortar specimens, as compared to those with Portland cement, promises a good behaviour in aggressive environment.

Keywords: Sulfoaluminate cement, low energy cement, high early strength, heat of hydration, porosity.

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INTRODUCTION

The interest on the calcium sulfoaluminate ($C_4A_3\check{S}$) based cements originated from the expansive effect, which can be obtained from the hydration of the $C_4A_3\check{S}$ phase and the subsequent ettringite formation[1–5]. The synthesis of expansive cements is based on this property. Expansive cement Type K contains suitable proportions of $C_4A_3\check{S}$, $C\check{S}$ and lime blended with portland cement.

Investigations of the hydration of sulfoaluminate cements have shown that appropriate proportioning of C_2S, C_4AF $C_4A_3\check{S}$ and $C\check{S}$ can produce cements with properties of rapid hardening, high early strength as well as exceptionally high strengths at later ages, with low or without any expansion[6–8].

On the other hand the study of the formation of a clinker with the above mineralogical compounds showed that it can be synthesized using usual raw materials in combination with industrial by-products[9,10], it can be sintered at temperatures about 150–200°C lower than Portland cement clinker[3,6,8,11], and it needs 50% lower grinding energy[12]; these are features that characterize the sulfoaluminate cements as low energy or energy saving cements.

For the above reasons, increasing attention has been paid to this type of cements, not only as expansive agents, but also for uses similar to Portland cement, especially where properties of rapid hardening and high early strength are needed.

In the present work a sulphoaluminate cement (SAC) was prepared in the context of a project aiming at the preparation of energy-saving cements having hydraulic behaviour similar to Portland cement (OPC). Based on the specific literature[3,7,8,12–16] a non-expansive clinker was synthesized[17]. The cement prepared was compared to OPC regarding its physical and mechanical characteristics.

EXPERIMENTAL DETAILS

The details of the preparation and composition of the sulfoaluminate clinker are described in a previous work[17]. The clinker was ground to cement in a laboratory Bond-mill without any addition of gypsum, requiring 2700 revolutions, in order to attain a specific surface (Blaine) 3770 cm^2/g. The same mill grinds an OPC clinker to cement of the same finess by 4000 revolutions.

The potential composition of the SAC and the OPC, which was used as reference, are given in Table 1.

TABLE 1 Composition of Sulfoaluminate and Ordinary Portland Cement.

Sulfoaluminate	Cement	Portland	Cement
$C_4A_3\check{S}$	20.1 %	C_3S	51.9 %
(β-C_2S	47.1 %	C_2S	18.8 %
$CaSO_4$	19.8 %	C_3A	6.4 %
C_4AF	13.8 %	C_4AF	11.9 %
		CaO_{free}	1.5 %

The dimensional stability of the prepared SAC was tested according to EN and ASTM standard methods and was found similar to OPC[17].

The setting time and the mechanical strength were measured according to EN Standard methods[18]. For the determination of the heat of hydration the AFNOR method was applied[19].

The development of the pore structure was studied by Hg intrusion (Carlo Erba Porosimeter 4000) in mortar specimens similar to those prepared for the mechanical strength measurements.

RESULTS AND DISCUSSION

Grindability. Considering the number of mill revolutions, required to grind the Sulfoaluminate clinker to cement, as a measure of its grindability, the prepared SAC needed 30% less grinding energy compared to an OPC, as it has also been confirmed from the literature[20].

Table 2 contains the physical and mechanical properties of two cements tested, the OPC as reference.

Setting time. The two tests for the determination of setting time showed that the SAC is generally a rapid setting cement. Initial and final setting times are both very strongly influenced by the amount of mixing water.

Strength. The mortar specimens, which were demoulded after 6 hours, exhibited rapid hardening properties, as well as early strengths higher than these of OPC (Table 2). After the first 24 hours the rate of strength development of SAC was lower than that of OPC. Thus, the compressive strengths at 28 and 90 days were lower than OPC. They were also lower than those expected[7]. This behaviour could be explained by the study of the hydration products[17]. The rapid hardening and the high early strength were attributed to the ettringite formation, while the lower rate of strength development and the lower

TABLE 2 Physical, mechanical and microstructural properties of SAC and OPC.

		SAC		OPC
Spe c. Gravity (g/cm^3)		3.05		3.14
Spec. Surface (cm^2/g) (Blaine)		3770		3790
Setting time				
Initial (min)	38		28	75
Final (min)	51		41	104
Mixing Water %	22.3		22.2	27.0
Compressive Strength (MPa)				
6 hours		9.75		
12 hours		15.4		6.0
1 day		17.4		16.2
2 days		19.9		27.3
7 days		30.9		42.5
28 days		34.1		53.5
90 days		35.6		63.4
Heat of hydration (J/g)				
		236		314
Total Pore Volume (mm^3/g)				
2 days		67.1		74.7
7 days		65.6		62.1
28 days		57.3		60.1
90 days		55.7		55.3

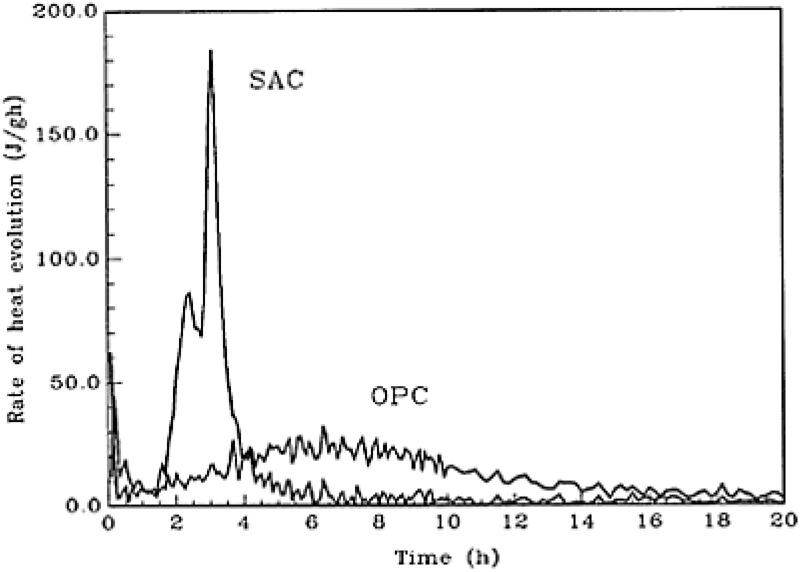

Figure 1: Rate of evolution of heat of hydration.

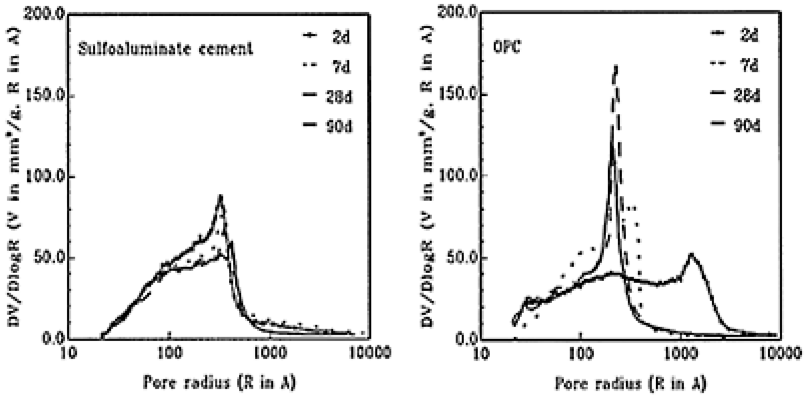

Figure 2, 3: Pore size distributions of hydrated SAC and OPC.

values at later ages to the low hydration degree of (β-C_2S.

The heat of hydration of SAC was lower than this of OPC (Table 2): However, the rate of heat evolution of SAC was higher than OPC in the first hours, so that the 98% of the total amount was liberated in the first 24h, while the corresponding time for the OPC was 72h (Fig. 1).

Porosity. The total pore volume of the SAC mortar specimens was significantly lower than that of OPC especially at the early hydration ages (Table 2). After 90 days of hydration the total pore volume of OPC specimens became similar to this of SAC. The pore size distributions of the SAC specimens (Fig. 2) showed that already after 2 days of hydration the most of the pores had diameters lower than 1000 A, while the OPC specimens had this size as mean pore diameter at the later ages (Fig. 3). After 28 and 90 days, hydration of the most OPC compounds resulted in the formation of a narrow distribution of pore diameters, while the corresponding curves of SAC formed a shoulder on the lower diameter side. It is considered that the latter could have a narrower form, if the hydration of SAC was more complete. All the above observations indicate that the studied SAC could exhibit a good behaviour in an aggressive environment.

CONCLUSIONS

The comparison in the hydraulic behaviour of the prepared Sulfoaluminate cement to an OPC showed that:

– It had shorter setting times (rapid setting cement).

– It developed high compressive strength in the first 24 hours, already from 6 hours on (rapid hardening and high early strength cement).

– It liberated most of its heat of hydration much earlier than OPC.

– It developed a low porosity structure already from an early hydration stage, indicating that it might have a good behaviour in aggressive environment.

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PHASE EQUILIBRIA IN THE CaO–Al_2O_3–SiO_2–H_2O SYSTEM IN RELATION TO BLENDED HAC CEMENTS

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ABSTRACT. The compressive strength of concretes made using high alumina cement can fall with time when kept under warm, humid conditions due to a process known as ‘conversion’. However, concretes made using mixtures of high alumina cement and pozzolanic or latently hydraulic materials have shown a trend of increased compressive strength when kept under similar conditions. These additions modify the hydration process, leading to the formation of strätlingite (also called gehlenite hydrate) as the main crystalline phase and reduce the effects of conversion. Mixtures of high alumina cement and ground granulated blastfurnace slag (ggbs) have been patented and given the trade name BRECEM.

The durability of BRECEM concretes will depend on the long term stability of gehlenite hydrate. A study of phase equilibria in the CaO–Al_2O_3–SiO_2–H_2O system has therefore been carried out at temperatures up to 60°C to determine the stability of gehlenite hydrate and the phase assemblages in which it forms. In this paper the results these studies are discussed in relation to results from studies of the hydration and durability of cements in which HAC was blended with ggbs, metakaolin and other latently hydraulic or pozzolanic materials.

Keywords: High alumina cement (HAC), ground granulated blastfurnace slag (ggbs), phase equilibria, strätlingite.

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INTRODUCTION

Concretes made from high alumina cement (HAC) have been found to lose strength with time, particularly in hot and humid environments, through a process known as 'conversion'. In the conversion process the initial crystalline products of hydration, the metastable calcium aluminate hydrates* CAH_{10} and C_2AH_8, are replaced by the stable calcium aluminate hydrate C_3AH_6 (hydrogarnet) and AH_3[1]. The conversion reactions occur extremely slowly at 5°C, but become more rapid as the temperature is increased and C_3AH_6 formation is almost immediate above 50°C. C_3AH_6 has a higher density (2530 kg m^{-3}) than either CAH_{10} (1730 kg m^{-3}) or C_2AH_8 (1950 kg m^{-3}) and so conversion is accompanied by a decrease in the solid volume and therefore an increase in porosity that can lead to a loss in strength[1,2].

The Building Research Establishment has been investigating cements in which HAC is blended with latently hydraulic materials such as ground granulated blastfurnace slag (ggbs) or pozzolanic materials such as pulverised fuel ash (pfa) and metakaolin[3–9]. Most of the work carried out so far has concentrated on mixtures of HAC with ggbs which BRE has patented and registered under the trade name BRECEM[3]. Concretes made by using BRECEM do not exhibit the loss of compressive strength that occurs in HAC concretes when kept at 38°C under wet conditions over a prolonged period of time. BRECEM concretes also perform well on storing in sulphate solutions and in aggressive marine and acid water environments[8,9].

The absence of strength loss in these blended HAC cements is a consequence of the modified chemistry on hydration caused by the high silica contents of the additions. When blended HAC cements are hydrated at ambient temperatures the metastable calcium aluminate hydrates form initially due to the hydration of the HAC component. However, as the pozzolanic or latently hydraulic material hydrates the concentration of silica in solution increases. A new hydrate, C_2ASH_8, known as strätlingite or gehlenite hydrate, then forms causing the calcium aluminate hydrates to redissolve substantially. As a result the proportion of the mix made up of the calcium aluminate hydrates will be much less than in HAC and so the effects of the conversion reaction on the strength of the concrete will be greatly reduced.

Majumdar and Singh [3] have studied the paste hydration of HAC with a number of additions other than ggbs. Those studied were gasifier slag, metakaolin, silica fume, Italian pozzolana, a high lime-containing pfa and a low lime-containing pfa. Bentsen et al[10] and Bayoux et al[11] have studied phase formation in pastes of calcium aluminate cements with silica fume. Collepardi et al[12] have studied the hydration of HAC with silica fume and fly ash. Several of these materials have the potential to stabilise HAC under hot and humid conditions. Gasifier slag, high lime pfa, metakaolin and silica fume appeared[3] to be the most successful after 1 year based on the compressive strength. C_2ASH_8 formed as a persistent phase (up to 1 year) in nearly all the mixes studied at both 20°C and 40°C. However, it was present[3] only

*Cement chemistry notation: C=CaO, A=Al_2O_3, S=SiO_2, H=H_2O, C=CO_2

in trace quantities in a 60 HAC: 40 pfa (low lime-containing) mix at 40°C after 180 days and was absent after 1 year.

The compressive strength of concretes made using HAC/ggbs mixes at 60°C has been reported [13] to be lower than expected. These results are consistent with those of a recent study carried out at BRE[14] in which the temperature, compressive strength and phase composition of a 1m^3 cube of BRECEM concrete cured under adiabatic conditions were monitored. The temperature of the cube was found to rise to a maximum of 58°C and the compressive strengths of cores taken from the cube were low. XRD showed that large amounts of C_3AH_6 were present. The results suggest that the HAC component hydrated quickly and, as a consequence of the temperature rise on hydration, the metastable hydrates quickly converted to C_3AH_6. C_2ASH_8 was not present initially, suggesting that it is either unstable or that it forms slowly at elevated temperatures. C_2ASH_8 was, however, found in large quantities at later ages when the temperature had fallen [8].

The stability of C_2ASH_8 is clearly a key factor in the long term performance of concretes made using blended HAC cements. If it is unstable it will eventually be replaced by another hydrate or hydrates. The main aim of the work reported here was to determine the stability of C_2ASH_8, the hydrates that form with it and the range of compositions and temperatures over which it forms. This has been done primarily by studying phase equilibria in the $CaO–Al_2O_3–SiO_2–H_2O$ system at temperatures between 5°C and 60°C. Such a study is valuable as the principal hydrates that form on hydrating HAC both on its own and in mixtures with ggbs and pozzolanas also form within the chemically simplified $CaO–Al_2O_3–SiO_2–H_2O$ system.

The solid phase compositions of some blended HAC cements have also been studied using accelerated ageing techniques. These techniques allow the hydrates that would normally be formed only after long periods of hydration to be determined in a relatively short period.

EXPERIMENTAL

Phase Equilibria Studies in the $CaO–Al_2O_3–SiO_2–H_2O$ System

Mixes with compositions in the $CaO–Al_2O_3–SiO_2$ system around those of idealised HAC/ggbs blends were prepared using CaO (prepared by heating $CaCO_3$ at 1200°C for 3 hours), silicic acid and aluminium hydroxide gel (AH_3). Some mixes were prepared using monocalcium aluminate (CA) instead of AH_3. 1 g of solid was used in 100 g of degassed deionised water. The mixes were sealed in polypropylene bottles and shaken continuously (by end over end tumbling at a rate of about 30 revs/min) at temperatures of 5°C, 20°C and 38°C. Tests at 60°C were carried out using an oven with samples shaken by hand at regular intervals. One mix, with an overall composition aimed to match that of C_2ASH_8 on hydration, was run at 90°C. The mixes were left for up to 18 months before filtering under nitrogen. The solids were dried and stored under vacuum prior to analysis by X-ray diffractometry.

Analysis was carried out using a Siemens D500 diffractometer using copper Kα radiation operating at 40 KV and 30 mA. Data were accumulated over one scan of 29 between 5° and 50°. Assignments of lines were made by comparison with JCPDS data files. The composition of the siliceous hydrogarnet ($C_3AS_nH_{6–2n}$, formed in preference to

C_3AH_6 when silica is available) was determined from the d-spacing corresponding to the 420 line (using quartz as an internal standard) [13]. Aqueous solutions were analysed as soon as possible after filtration for calcium and aluminate ions using atomic absorption spectroscopy, and for silicate ions using a molybdenum blue photometric method. The results of the solution analyses have been used in a modelling study of parts of the CaO–Al_2O_3–SiO_2–H_2O system [15].

HAC/ggbs and HAC/Metakaolin Blends

Mixtures of HAC with ggbs or metakaolin (see Tables 1 and 2) were mixed with a large excess of water (waten:solid ratio=50) and left to approach equilibrium for several months at temperatures of 5°C, 20°C, 50°C and 90°C whilst being constantly shaken. Prior to analysis the equilibrated mixes were filtered by vacuum under nitrogen and dried over P_2O_5. The dry solids were analysed using X-ray diffractometry (XRD) in order to determine the hydrates present. Durability studies on HAC/metakaolin concretes have given promising results[2].

RESULTS

The CaO–Al_2O_3–SiO_2–H_2O System

C_2ASH_8, the principal crystalline hydrate formed in BRECEM, was detected over a wide range of mix compositions at all temperatures studied and was present in both stable and metastable phase assemblages[15] summarised below. The stable assemblages formed at all temperatures, particularly at 38°C, whereas the metastable assemblages were found only at 5°C and 20°C.

Stable assemblages	**Metastable assemblages**
C_2ASH_8/hydrogarnet(l)/AH_3	C_2ASH_8/CAH_{10}/AH_3
CH/hydrogarnet(1)/C-S-H	C_2ASH_8/C_2AH_8/CAH_{10}
C_2ASH_8/hydrogarnet(1)/hydrogarnet(2)	C_2ASH_8/C_4AH_{19}/C_2AH_8
C_2ASH_8/hydrogarnet(2)/C–S–H	C_2ASH_8/C–S–H/C_4AH_{19}
C–S–H/hydrogarnet(1)/hydrogarnet(2).	C_4AH_{19}/C–S–H/CH.

Hydrogarnet(1) and hydrogarnet(2) were approximately $C_3AS_{0.3}H_{5.4}$ and $C_3AS_{0.9}H_{4.2}$ respectively. These studies suggest that C_2ASH_8 is a stable phase at temperatures up to 38°C, supporting the results of durability studies that have shown that BRECEM concretes maintain their strength over long periods of time.

Gehlenite hydrate was formed in pure oxide mixes at 60°C after 16 months and is likely to be a stable phase at this temperature. However, it was absent from all mixes studied after 4 months, indicating that its formation at this temperature is slow. Siliceous hydrogarnets were present in all the mixes studied and are also likely to be stable phases at 60°C. The hydrogarnet peak positions were not accurately determined, but appear to be consistent with the compositions determined at the lower temperatures. AH_3 (gibbsite) is formed in substantial quantities and is likely to be a stable phase.

C–S–H gel was present in some mixes. There was some evidence of the formation of crystalline calcium silicate hydrates at 60°C; either C–S–H(1) or a tobermorite-like material. However, it was not possible to make a definite assignment from the XRD peaks. The phase assemblage $C_2ASH_8/C_3AS_{0.3}H_{5.4}/AH_3$ appears to be stable at 60°C. The assemblage $C_3AS_{0.3}H_{5.4}/C_3AS_{0.8}H_{4.4}$/C–S–H may also be stable at 60°C. It was detected after 4 months but after 16 months $C_3AS_{0.8}H_{4.4}$ had been replaced by C_2ASH_8. Both these assemblages were also found to be stable at 38°C.

C_2ASH_8 was not formed in the single mix run at 90°C. A siliceous hydrogarnet with a composition of approximately $C_3AS_{0.3}H_{5.4}$ together with a zeolite phase with an XRD pattern similar to those of thomsonite ($NaCa_2Al_5SiO_{20}.6H_2O$) and scolecite ($CaAl_2Si_3O_{10}.3H_2O$) were identified by XRD. A calcium silicate hydrate phase with an XRD pattern similar to that of hillebrandite (C_2SH) may also have been present.

Table 1. Hydrates forming in HAC/ggbs mixes.

T/°C	HAC:ggbs	Age/d	AH_3	CAH_{10}	C_2ASH_8	hydrogarnet	CSH	Others
5	60:40	232		✓	✓			C_4AcH_{11}
5	40:60	232		✓	✓		?	C_4AcH_{11}
20	60:40	232	✓		✓	✓[1]		
20	50:50	230			✓		✓	
20	40:60	231			✓		✓	C_4AcH_{11}, $C_4Ac_{0.5}H_{12}$
50	60:40	232	✓		✓	✓[1]		
50	50:50	234			✓	✓[1]		C_4AcH_{11}, ettringite
50	40:60	234			✓	✓[1]	✓	
90	60:40	276				✓[2]		
90	50:50	276				✓[2]		unidentified
90	40:60	276				✓[2]		unidentified

Accelerated hydration (w/c=50, constant mixing) used in all tests.

1. Hydrogarnet(1) present. 2. Hydrogarnet(1) and hydrogarnet(2) present.

HAC/ggbs and HAC/metakaolin blends.

The hydrates found in accelerated ageing tests on HAC/ggbs mixes in the proportions 60:40, 50:50 and 40:60 are shown in Table 1. They were generally consistent with those found in BRECEM concretes [3]. C_2ASH_8 was present in large amounts at temperatures up to 50°C, with CAH_{10} or hydrogarnet, depending on the temperature. C_4AcH_{11} or hydrotalcite (these two phases are difficult to distinguish by XRD) was also present in some mixes. Ettringite was present in small quantities in the 50 HAC:50 ggbs mix at 50°C. The 40 HAC:60 ggbs mix at 50°C contained some C-S-H gel. Mixtures of HAC with metakaolin (Table 2) contained CAH_{10}, C_2ASH_8 and some C_4AcH_{11} or hydrotalcite at 5°C and 20°C. At 50°C hydrogarnet, C_2ASH_8 and AH_3 (gibbsite) were present. These results are consistent with the studies on the $CaO–Al_2O_3–SiO_2–H_2O$ system. Gehlenite hydrate was not present in any mix at 90°C, supporting the view that it is not likely to be a stable phase at this temperature. Two hydrogarnets were present in all three mixes at 90°C. However, the XRD peaks for the two hydrogarnet phases were close preventing any precise compositional information being derived. A third phase was also present in the 50 HAC:50 ggbs and 40 HAC:60 ggbs mixes. It was characterised by an XRD peak at 6.1 Å. This phase has not been identified, but it may be a crystalline calcium silicate hydrate.

Table 2. Hydrates formed in HAC/metakaolin (MK) blends

T/°C	HAG:MK	Age/days	AH_3	CAH_{10}	C_2ASH_8	hydrogarnet	CSH	Others
5	50:50	226		✔	✔			C_4AcH_{11}
20	50:50	226		✔	✔			C_4AcH_{11}
50	50:50	223	?	✔	✔			
50	75:25	223	✔		✔	✔		C_4AcH_{11}?

Accelerated hydration (w/c=50, constant mixing) used in all tests.
? indicates that the phase may be present.

DISCUSSION

Phase equilibria studies in the context of blended HAC cements

The results of phase equilibria studies in the $CaO–Al_2O_3–SiO_2–H_2O$ system show that C_2ASH_8 is a persistent phase in the temperature range 5°C to 60°C. It forms a stable phase assemblage with hydrogarnet and AH_3, the principal stable products of HAC hydration, and also forms assemblages with the metastable hydrates CAH_{10} and C_2AH_8, the initial products of HAC and BRECEM hydration. C_2ASH_8 has been found to be the main crystalline hydrate at temperatures up to 38°C in most of the blended HAC cements that have been studied so far[3] although the amount forming in a hydrated cement will depend on the cement composition. The results of the stability studies therefore support the view that concretes prepared using blended HAC cements will maintain their strength in the long term, at least at ambient temperatures. The results are also consistent with studies[3] on BRECEM concretes in which C_2ASH_8 was present in substantial quantities for at least 5 years at temperatures up to 38°C.

The development of C_2ASH_8 in pure oxide mixes at 60°C has been found to be very slow. Its formation in BRECEM concretes has also been slow at elevated temperatures. At its present state of development, therefore, BRECEM may not be suitable for use in all applications for mass concrete, although this is an area in which HAC is also unsuitable. BRECEM could be used in mass concrete applications where slower strength development is allowed, but may not be suitable if rapid strength development was a priority[11]. However, promising results have been obtained for precast BRECEM concretes that were sprayed after demoulding to control the temperature rise on hydration. Strength development was good and C_2ASH_8 was detected in substantial quantities at early ages[8]. It may be possible to accelerate its formation at this temperature by using a source of silica that is more reactive than ggbs. If so it may be possible to improve the compressive strength development of mass BRECEM concretes.

Damidot and Glasser[16] have carried out modelling studies on the $CaO–Al_2O_3–SiO_2–H_2O$ system at 25°C and their results were consistent with those reported here. Damidot[17] has also extended this study to show that C_2ASH_8 is more stable in the presence of sulphate and carbonate ions than is C_3AH_6. This is consistent with the observed increased sulphate resistance observed for BRECEM concretes.

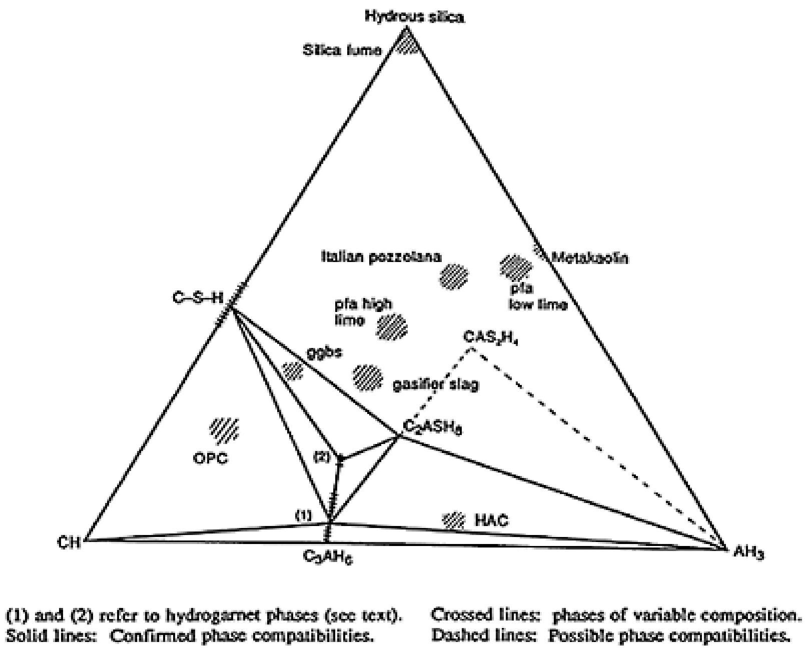

Figure 1. Phase compatibility in the $CaO–Al_2O_3–SiO_2–H_2O$ system and the compositions of cementitious components.

Figure 1 shows the stable and metastable phase assemblages found in the study of the $CaO–Al_2O_3–SiO_2–H_2O$ system at temperatures up to 38°C, projected on to the $CaO–Al_2O_3–SiO_2$ surface. Phases that were found to be compatible in the experimental work on the $CaO–Al_2O_3–SiO_2–H_2O$ system are linked using solid lines. The figure also shows the 'idealised' compositions of HAC, together with those of potential additions. The compositions of blended cements will lie between those of HAC and the particular addition. Blended HAC cements cannot be accurately modelled by the $CaO–Al_2O_3–SiO_2$ system as they contain other oxides in significant quantities. Other hydrates such as calcium carboaluminate hydrates and hydrotalcite may also form. Figure 1 is therefore a simplification. Despite this, Figure 1 should allow the main equilibrium products formed on hydrating an HAC blend to be predicted. These will be the 3 hydrates forming the triangle in which the overall mix composition falls. The relative amounts will be inversely proportional to the distance between the hydrate and mix compositions. As HAC hydrates more rapidly than the additions the hydrated phases present will initially be the calcium aluminate hydrates and AH_3. C_2ASH_8 will form as the addition hydrates.

The extent to which the addition hydrates will be dependent on the availability of water and the reactivity of the silica-containing addition. For an addition to be viable it must increase the availability of silica during the hydration reaction and must result in the formation of substantial amounts of C_2ASH_8 in order to limit the effects of the conversion reaction as discussed above. The amount of C_2ASH_8 forming will be greater in blends with overall compositions that are closest to that of C_2ASH_8 in Figure 1. Therefore blends of HAC with ggbs, gasifier slag or pfa (USA) should all contain large amounts of C_2ASH_8, depending on the relative proportions of the two constituents. HAC blended with metakaolin and pfa (UK) would be expected to contain less C_2ASH_8 as the compositions of mixes lie further away from the C_2ASH_8 composition although it does not automatically follow that the compressive strengths of concretes made using these blends would be low. Collepardi et al[12] have shown that the addition of low lime-containing pfa to HAC is not advantageous in reducing the effects of the conversion reaction. However, studies on concrete cubes made using HAC and metakaolin blends have shown promising results[3].

The phases formed[3] on hydrating 50:50 blends of HAC with the additions listed in the introduction were CAH_{10} or a siliceous hydrogarnet, C_2ASH_8 and, in some cases, AH_3. The presence of CAH_{10} and hydrogarnet in the same mix would indicate that the conversion process was still occurring leading to the formation of the stable assemblage C_2ASH_8/hydrogarnet/AH_3. However, from Figure 1 it is apparent that the compositions of 50:50 blends of HAC with silica fume, Italian pozzolana, metakaolin and both types of pfa may fall outside the composition triangle defined by the stable phase assemblage C_2ASH_8/hydrogarnet/AH_3 (although variations in the compositions of the constituents and the presence of other oxides such as Fe_2O_3 and MgO will affect the exact position of the compositions). They lie in a triangle defined by C_2ASH_8, AH_3 and a zeolite-type phase such as gismondine (CAS_2H_4). The calcium aluminate hydrates that form in these blends[3] may therefore eventually disappear, although this process may be extremely slow at ambient temperatures. Studies of the hydration of HAC/metakaolin blends described above showed that hydrogarnet (or CAH_{10} at lower temperatures) was present after several months, together with C2ASH8 and AH_3. Zeolites have been found to form[18,19] in blended Portland cements at temperatures below 90°C.

The compositions of the 50 HAC: 50 ggbs and 40 HAC: 60 ggbs BRECEM mixes are within the triangle defined by the phases C_2ASH_8, C–S–H and a hydrogarnet with the approximate composition $C_3AS_{0.9}H_{4.2}$. However, because of the relatively slow hydration of the ggbs component the phases detected in BRECEM concretes after 5 years are C_2ASH_8, AH_3 and a hydrogarnet with an approximate composition of $C_3AS_{0.3}H_{5.4}$. As C_2ASH_8 is the predominant phase in BRECEM mixes any changes in the minor phases, if they occur at all, would probably not be significant. Such changes would not be expected to occur in the 60 HAC: 40 ggbs mix for compositional reasons. The accelerated ageing study of HAC/ggbs blends described in section 3 showed that C–S–H formed in 40 HAC:60 ggbs blends at 50°C.

CONCLUSIONS

C_2ASH_8, the principal phase formed on hydrating blended HAC cements, is a persistent phase in the $CaO–Al_2O_3–SiO_2–H_2O$ system at temperatures up to at least 60°C. This result is consistent with durability studies on BRECEM concretes in which the compressive strength is maintained for at least 5 years under wet conditions at 38°C. Other blended HAC cements have also given promising results in durability studies after 1 year but their long term behaviour needs to be confirmed. The increased stability of BRECEM cements on exposure to aggressive environments can also be assigned to the stability of C_2ASH_8.

The low compressive strength of BRECEM concretes cured under adiabatic conditions is probably associated with the slow formation of C_2ASH_8 at temperatures of about 60°C. C_2ASH_8 is, however, likely to be stable at this temperature. It may be possible to accelerate its formation at 60°C if a suitably reactive source of silica can be identified. C_2ASH_8 is probably unstable at 90°C.

Comparisons of the compositions of blended HAC cements with phase compatibility diagrams for the $CaO–Al_2O_3–SiO_2–H_2O$ system show that HAC blended with ggbs, gasifier slag or high lime-containing pfa should produce the most C_2ASH_8 on hydration. Hydrogarnet (or one of the metastable calcium aluminate hydrates) and AH_3 would also be present. HAC blended with metakaolin or low lime-containing pfa would be expected to contain less C_2ASH_8 on compositional grounds, and it is possible that a zeolite-like phase may eventually be formed, especially at high addition levels. However, accelerated ageing studies on HAC/metakaolin blends have not shown the presence of any new phases.

BRECEM may eventually be used as a specialist repair material for existing concrete structures or in structural concrete elements such as precast piles, prestressed beams, tank sections and wall/floor slabs. However, its viability for use will depend on the costs of basic materials[8] and its acceptance via standards and codes of practice.

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HIGH PERFORMANCE CONCRETE CONTAINING MODIFIED RICE HUSK ASH

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ABSTRACT. The purpose of this research study was to investigate experimentally the mechanical behavior of high performance concrete containing modified rice husk ash, hereinafter referred to as MRHA. The main parameters were the type and percentage of MRHA and type of superplasticizer. The dosage of superplasticizers, aggregate-cement ratio and sand-aggregate ratio by weight were kept constant. The mechanical properties of high performance concrete which were investigated include the compressive and flexural strengths, and slump retention characteristic. The results showed that the highest compressive strength was obtained when 30% of MRHA type 3 was used to replace ordinary Portland cement. The strength and slump retention of concrete were considerably reduced when the percentage of MRHA replacement exceeded 50% by weight.

Keywords: High performance concrete, mechanical properties, modified rice husk ash, superplasticizer.

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INTRODUCTION

In the developing countries, considerable efforts have been directed towards the utilization of indigenous and waste materials. Not surprisingly there has been interest in the use of rice husk as a building material and limited success has been achieved with the use of the husk in particle board and cement-husk building block [1]. A more promising area of interest however has been the use of the rice husk ash (RHA) either as pozzolan [2] or in the manufacture of cementitious materials [3].

The present study is aimed at investigating the mechanical properties of high performance concrete in which the matrix contains modified rice husk ash. The main parameters are the type and percentage of MRHA. The percentages of MRHA used as cement replacement vary from 10 to 60% and the percentage of MRHA used as cement addition is 10%. The dosage of superplasticizer is kept constant at 2.4% of cementitious material by weight. The aggregate-cement and sand-aggregate ratios by weight are kept constant and equal to 3.25 and 0.40, respectively. The slump is maintained at 20 ± 2 cm. The study is undertaken with the primary aims of determining the followings:-

(i) The effects of different types and percentage replacements of MRHA on the compressive strength at different ages and 28-day modulus of rupture of high performance concrete and slump retention over a duration of 90 minutes.

(ii) Optimum mix of high performance concrete containing MRHA based on the mechanical properties obtained.

EXPERIMENTAL DETAILS

Preparation of Modified Rice Husk Ash

The major characteristics of RHA are its high water demand and low degree of fineness as compared with condensed silica fume. Another important problem is how to disperse RHA particles uniformly in the mix. The recommended grinding time for RHA was 45 minutes [4,5]. The degree of fineness achieved by this was not sufficient to make high performance concrete. To solve these problems, rice husk ash must be ground in the grinding machine for a long duration of time in order to achieve a very high degree of fineness and absorbed a superplasticizer namely MIGHTY 150 to reduce the water requirement of RHA. A grinding duration of 1 hour and 15 minutes is proposed in this study and this degree of fineness will be maintained throughout the whole testing program. A dispersing agent was also used to disperse RHA particle uniformly in slurry form. The modified rice husk ash is classified into 3 types namely, MRHA type 1, MRHA type 2 and MRHA type 3.

MRHA Type 1

RHA, water and a dispersing agent were mixed together by the mixer. The RHA and a dispersing agent were initially fed into the mixer followed by a portion of the required water. The mixing duration was 1 minute. The RHA: water ratio was 1:1. The dosage of a dispersing agent was 0.2% of RHA-water mixture by weight.

MRHA Type 2

RHA, water, a dispersing agent and MIGHTY 150 were mixed together by the same mixer. The RHA and a dispersing agent were initially fed into the mixer. The required water and MIGHTY 150 were subsequently added into the mixer. The mixing duration was 2 minutes. The RHA: water ratio was 1:1. The dosages of a dispersing agent and MIGHTY 150 were 0.2% of RHA-water-MIGHTY 150 mixture by weight and 3% of RHA by weight, respectively.

MRHA Type 3

RHA, water, a dispersing agent and MIGHTY 150 were mixed together by the same mixer. The mixing procedure was similar to that of MRHA type 2. The RHA: water ratio was 1:1. The dosages of a dispersing agent and MIGHTY 150 were 0.2% of RHA-water-MIGHTY 150 mixture by weight and 3% of RHA-water mixture by weight, respectively.

Preparation of Other Constituent Materials

(a) Rice Husk Ash: It was obtained by burning rice husk in the ferrocement incinerator and grinding in the grinding machine [4,5].

(b) Portland Cement: Ordinary Portland cement Type I was used.

(c) Mixing Water: Ordinary tap water was used.

(d) Fine Aggregate: Natural river sand passing ASTM sieve number 3/8 in. was used.

(e) Coarse Aggregate: The maximum size of aggregate used was 10 mm.

(f) Superplasticizer: The superplasticizer namely MIGHTY 150 supplied by Mineral and Chemical (Thailand) Co., Ltd. was mainly used in the testing program.

(g) Fly Ash: Three types of fly ash namely MM-FA supplied by the Mae Moh Electrical Power Plant, HK-FA supplied by Thai Master Builders Co., Ltd. and Siam-FA supplied by Siam Cellulose Co., Ltd. were used.

RESULTS AND DISCUSSION

Properties of Pozzolans

It can be seen that the three different pozzolans designated as RHA, MM-FA and HK-FA composed of different amounts of oxides. RHA before modification contained very high amount of SiO_2, and little amount of Fe_2O_3, CaO, MgO and SO_3. Because of the high content of SiO_2, which plays an important role on pozzolanic properties; and its high water requirement, it was evident that RHA met the requirements of pozzolan class N as shown in Table 1. When comparing with MM-FA and HK-FA, it was found that the SO_3 content of MM-FA were much higher. MM-FA had higher SiO_2 and Al_2O_3 contents but the Fe_2O_3 and CaO contents were higher. This may be due to the different kinds of the raw materials and the combustion processes by which pozzolan is formed. It was also observed that RHA possessed lower specific gravity than fly ash. The reason is that RHA has very high content of silica.

The fineness of RHA passing no. 325 sieve and pozzolanic activity index had larger values than fly ash. This means that the particle size of RHA is smaller than fly ash. Because RHA particles are not spherical, RHA cement requires more water to produce the same consistency as that of fly ash cement.

High Performance Concrete

Effect of MRHA Type and Percentage Replacement

i) Strength

The results on compressive strengths which were summarized and shown in Figure 1 indicated that among the three types of MRHA investigated, MRHA type 3 was the most effective pozzolan and the optimum percentage replacement for all types of MRHA was 30%. It can be observed that the compressive strengths at ages of 3 and 7 days of MRHA concrete were lower than those of the control concrete. This is due to the fact that MRHA reacted slowly with calcium hydroxide liberated from the reaction between the C_3S and C_2S compounds and water to form calcium silicate hydrates. Thus, the early compressive strength of MRHA concrete were found to be lower than that of the control concrete. This phenomenon is similar to fly ash concrete. However, at ages of 28 and 56 days, MRHA concrete having MRHA replacement up to 30% showed compressive strength higher than the control concrete. At MRHA replacement 40%, the compressive strength at all ages was found to be less the same as those of the control concrete. However, if the percentage replacement of MRHA exceeded 50%, the compressive strength at all ages was considerably reduced. It was also indicated that the highest compressive strength was obtained when using 10% MRHA as cement addition. Regarding the

Table 1 Chemical and physical properties of pozzolans as compared with those of ASTM requirements

CHEMICAL PROPERTIES	ASTM C618–91 CLASS			TYPE OF POZZOLAN		
	N	F	C	RHA	MM-FA	HK-FA
$SIO_2+AL_2O_3+FE_2O_3$ (MIN, %)	70	70	50	92.28	59.97	80.91
SO_3 (MAX, %)	4	5	5	0.18	1.64	0.34
MGO (MAX, %)	5	5	5	0.18	3.52	1.01
NA_2O (MAX, %)	1.5	1.5	1.5	0.03	1.12	0.20
LOI (MAX, %)	10	12	6	3.67	0.27	2.59
PHYSICAL PROPERTIES						
FINENESS: AMOUNT RETAINED WHEN	34	34	34	0.36	17.35	6.08

WET-SIEVED ON NO. 325 (MAX, %)						
POZZOLANIC ACTIVITY INDEX WITH PORTLAND CEMENT AT 28 DAYS (MIN, %)	75	75	75	87	86	73
WATER REQUIREMENT (MAX, %)	115	105	105	109	89	95
SPECIFIC GRAVITY		2.10–2.40*		2.082	2.409	2.263
BLAINE FINENESS (CM^2/GM)		2460–3220**		7684	2739	3926
MOISTURE CONTENT (MAX, %)	3	3	3	0.67	0.02	0.06

* KOKUBU [6] ** DAVIS ET AL [7]

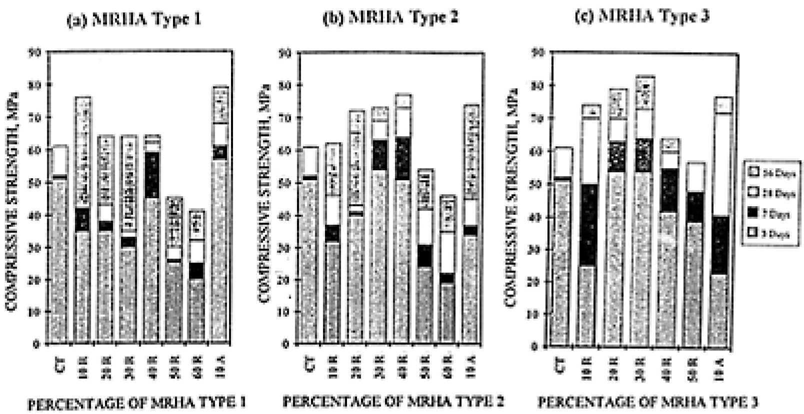

Figure 1 Variation of compressive strength at different ages of high strength concrete containing different types and percentages of modified rice husk ash

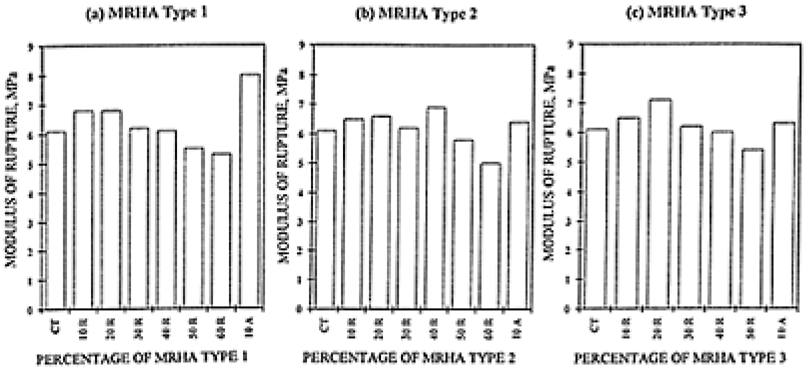

Figure 2 Variation of modulus of rupture at 28 days of high strength concrete containing different types and percentages of modified rice husk ash

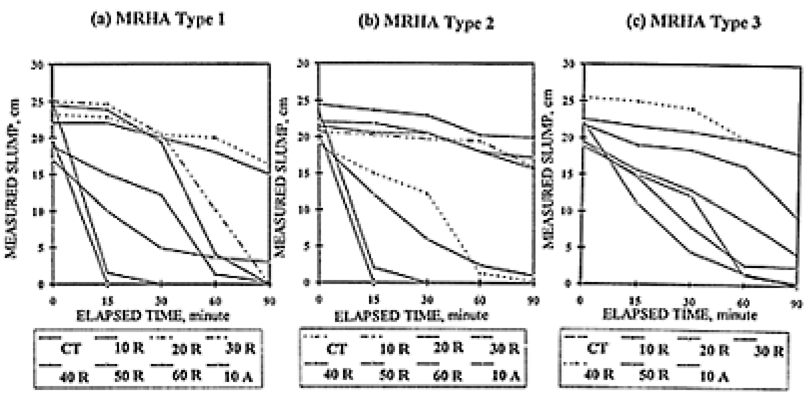

Figure 3 Relationship between slump and elapsed time of fresh concrete containing different percentages of MRHA

testing results on the modulus of rupture at 28 days shown in Figure 2, the trend was observed to be more or less the same as that of compressive strength.

ii) Workability

Regarding the properties of fresh concrete containing MRHA, the relationships between slump and elapsed time of fresh concrete containing different percentages of MRHA types 1, 2, and 3 are shown in Figure 3. The results indicated that for MRHA types 1 and 2, the slumps of concretes containing 50 and 60% MRHA replacements were sharply reduced within a period of 15 minutes after mixing whereas for fresh concretes containing 10 to 30% MRHA replacements, the workabilities of concretes were maintained over a period of 90 minutes. For MRHA type 3, the slump retention of the fresh concrete was better than the other two types of MRHA when 50% of MRHA replacement was used. For all types of MRHA, the percentage replacements below 40% resulted in significantly improved the slump retention performance of fresh concrete. It can be noted that for MRHA replacements of 10 and 20%, the slump losses were found to be only 15 to 20% over the duration of 90 minutes. It is evident that the replacements of cement by MRHA (10 and 20%) significantly improved the slump retention characteristics of fresh concretes as compared to the control concrete. This may be due to the effect of dispersing agent which was used in the modification of rice husk ash. An important point which should be emphasized is that high amount of MRHA replacement resulted in a poor slump retention characteristic of fresh concrete.

CONCLUSIONS

High performance concrete can be developed by replacing cement with modified rice husk ash which was obtained by premixing rice husk ash to water, a dispersing agent and superplasticizer in slurry form. The highest compressive strength was obtained when 30% of MRHA type 3 was used to replace ordinary Portland cement. The strength and slump retention of concrete were considerably reduced when the percentage of MRHA replacement exceeded 50% by weight.

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SLAG ALKALINE POLYMER CEMENT CONCRETES

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ABSTRACT. In the present paper the processes of modification of slag alkaline binders (SAB) and concretes (SAC) with polymer additives (latex, petrolatum, etc) with the aim of increasing their special properties in particular damping properties are examined. The rational methods of adding the optimum quantity of polymer additives in their composition are determined. The polymer slag alkaline concretes with increased damping capability of 20–25% compared to the traditional slag alkaline compounds were achieved. Besides, polymer modification enables to increase the flexural strength (R_f) by 10–15% compared to the control, while keeping the compressive strength (R_c) unaltered. Polymer slag alkaline concretes can be used as materials for seismic construction of foundation, high-way engineering containers and precision equipment.

Keywords: Slag Alkaline Cement and Concrete, Latex, Emulsion Petrolatum, Porosity, Logarithmic Decrement of Damping, Medium Radius of Pores.

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INTRODUCTION

Slag alkaline cement (SAC) and concretes on their base possess high physical-mechanical and some special properties [1,2]. Further improvement of properties of these materials may be achieved by their modification with additives of organic and mineral origin [3,4,5].However, low strength of the Portland cement matrix restricts the use of these materials. This paper reviews the principle of latex and petrolatum modification of slag alkaline cements and concretes and discusses their typical and special properties.

EXPERIMENTAL DETAILS

Materials and Methods. Slag alkaline cements and concretes on their base have been chosen as basic materials for investigation. Slag alkaline cemens (SAC) are composed of ground blast furnace slag with M_b=1.2. Quartz sand with a gradation factor 1.7 and crushed granite with particle size 5–10 mm were used as aggregates.

The additives of petrolatum emulsion and latex in a form of stabilized dispersion (50%—solids) were used for modification. The addition of latex into the slag alkaline cements was done in two ways: I—together with alkaline solution; II—while grinding the slag. The additive of petrolatum emulsion was added when grinding the slag. The testing procedure for determining. The main physico-mechanical properties didn't differ the from conventional procedurer appied to the traditional cementitious materials.

Experimental Part. Discussion of Results. It is known, that the modification of portland cement materials with the latex dispersions is hindered by the unstabiliby in the dispersion medium of hardened binder [5,6]. These factors, certainly, take place under the modification of slag alkaline cements. Hence it is, necessarily to take into account the high alkality of the medium (pH=12…13) of these cements. The modification of SAC can be attained by the use of stabilized latex, mixed for 2–3 minutes in a high-speed mechanical mixer just before application, as shown by our research.

The influence of quantity and method of adding latex, on the alkaline solution to slag ratio, setting time and strength characteristics of these cements were determined. The results are presented in Table 1.

The addition into cement's composition of the latex in quantity to 2.5% (dry state) does not depend on the methods of application and has no influence on the strength of these cements. A further increase in content of latex (to 10%) results in a strength decline as compared with that of control. The application of latex dispersions allows to extend slightly the setting time. This is, evidently, a result of the formation of polymer films on slag grains, which prevent its hydration. It is demonstrated also the decrease of alkaline solution to slag ratio from 0.28 to 0.24 accounting to a plastisizing effect of latex and explained by the presence of surface active substances (SAS) the within compositions of their stabilizers. The kinetic change of strength of modified slag alkaline cements hardened during different conditions for one year was revealed. The study's results are given in Figures 1 and 2.

As is follows from the obtained data the character of strength change of latex-modified cements is identical to the change of strength of the control. At equal quantities of latex and similar curing conditions the strength characteristics of the latex-modified cements treated by the first method are higher than those of cements treated by second method. The flexural strength of the latex-modified slag alkaline cements is over 10…15% higher than that of the control. This tendency iprevails for one year of hardening.

Table 1 Influence of amount and method of adding latex on physical-mechanical properties of slag alkaline cement

NOS OF COMPOSITION	LATEX CONTENT, % (dry state)	ALKALINE SOLUTION/SLAG RATIO	SETTING TIME, min		COMPRESSIVE STRENGTH, MPa
			Initial	Final	
1	–	0.28	8	13	94
2	1.0	0.26	10	15	92
3	2.5	0.25	12	17	90
4	5.0	0.25	12	17	85
5	10.0	0.24	12	18	70
6	1.0	0.26	12	17	90
7	2.5	0.26	15	20	88
8	5.0	0.25	14	20	82
9	10.0	0.25	15	22	75

* 1—control composition

** 2–5 compositions (possessing latex bv I method d)

*** 6–9 compositions (possessing latex by II method)

A method of mathematical planning of experiments was used to determine an optimum content of latex in slag alkaline concrete (it is found as 1% dry mass of slag) and alkaline solution to slag ratio as 0.36, at which the maximum compressive strength 94 MPa and flexural strength 9.5 MPa may be achieved.

The latex and emulsion petrolatum modifications of slag alkaline cements enhance considerably their damping properties. Table 2 indicates the results of evaluation of damping properties, porosity and medium effective radius of the latex—and petrolatum—modified cement and the control composition. The growth of LDD value is evidently connected with a three-stage prosess of structure formation and occurence of two levels of the pore structure. At the initial stage of structure formation a great growth of value of th medium radius of coarse capillary pores is revealed. An important role in formation of total level of pores is attributed to their filling up with the polymer, since its high level of viscous friction favours a fast damping. It is demonstrated that the LDD of the modified slag alkaline cements after long-term hardening tends to grow. So, the LDD of the compositions 2 and 3 tested for the second time after 6 months had increased by 17…20% as compared with that of the control. The content of polymer 2…2.5% of slag mass yet keeps the continuity of gel in structure. The physical-mechanical and special properties of modified slag alkaline concretes are determined using compositions from the latex-modified slag alkaline cements and quartz sand taken in a ratio 1:3 and 1:2:4 (cement: sand: coarse aggregate).

The shrinkage deformation of the latex-modified slag alkaline mortars is higher than that of control composition (1.2…1.3 mm/m and 0.9 mm/m, respectively) (Figure 3, curves 1, 2 and 3). The introduction into concrete of coarse aggregate with fractions 5–10 mm or reinforcing with coarse basalt fibres (d=100…200 mcm) is considerably decreased the shrinkage deformation (Figure 3, curves 4, 5). It this case, the most considerable shrinkage deformation are observed during 30…50 days, which stabilized at 60 days.

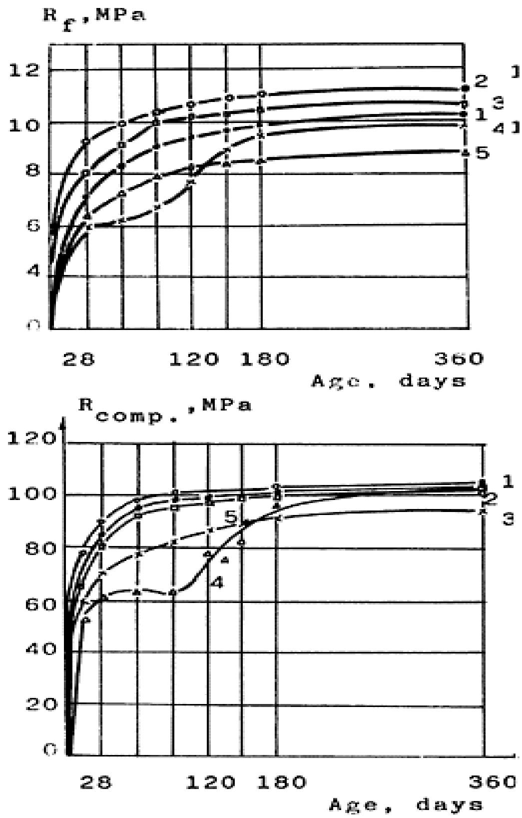

Figure 1 Change in Flexural (a) and Compressive (b) Strengths of Latex Modified SAC

Curing conditions normal.

1—without latex (control);

2–1%; 3–2.5% of laex (1 way);

4–1%; 5–2.5% of latex (2 way)

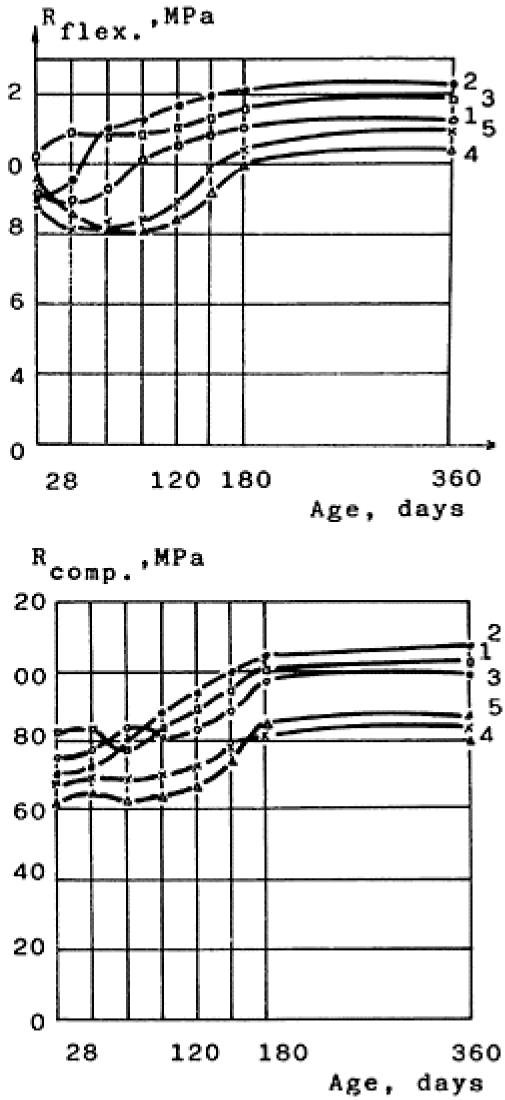

Figure 2 Change in Flexural (a) and Compressive (b) Strength of Latex Modified SAC

Curing conditions—steam curing followed by hardening under normal conditions. Designations as in Figure 1

The use of modification additive of emulsion petrolatum, in distinction from latex, ixpresses reduction moisturing deformation of shrinkage in concrete. The shrinkage of the petrolatum-modified slag alkaline mortars (Figure 3, curve 6) are lower, than that of the control composition and are found to be at a level of shrinkage of concrete with coarse aggregate size 5–10 mm. The increase in the linear dimensions is found when the spesimen were cured in water, it reaches a value of 0.4 mm/m on 150 days (control) and 0.6…0.65 mm/m (latex-modified specimen). The absolute values of the swelling are appr. 2.0…2.5 times less than those of shrinkage of the similar compositions.

Table 2 Damping properties of modified slag alkaline cements and concretes

NOS OF COMPOSITION	TYPE AND CONTENT OF POLYMER ADDITIVE, %	LOGARITHMIC DECREMENT OF DAMPING		POROSITY, %		MEDIUM EFFECTIVE PORE RADIUS, $r_{med}\cdot10^{-8}$	
		Cement Poste	Concrete	Cement Paste	Concrete	Cement Paste	Concrete
1	–	0.038	0.026	23.0	10.0	8.8	8.3
2	latex, 1	0.054	0.044	22.7	10.8	8.6	6.8
3	latex, 2.5	0.064	0.052	21	11.0	8.7	6.6
4	emulsion petrolatum	0.060	0.045	21.2	10.4	8.7	6.4
2*	latex, 1	0.056	0.46	22.0	10.6	8.5	6.7
3*	latex, 2.5	0.076	0.070	19.0	10.0	8.0	6.0

* Compositions 2 and 3 were tested for the second time after 6 months.

The damping properties of the slag alkaline concretes depend first of all on the similar characteristics of the slag alkaline cements. Table 2 is presented the investigation results of LDD of the modified slag alkaline concretes. A decline in the absolute value of LDD of the slag alkaline concrete as compared with that of the slag alkaline cements is attributed to a saturation of the bulk of the material with quartz sand characteristic of low LDD values (LDD of quartz sand is 0.005). The damping properties (evaluated by the LDD) of the latex-modified slag alkaline concretes are found to be at a level of the similar characteristics for polymer concretes.

Table 3 Crack resistance and coefficient of softening(C_s) of the latex-modified slag alkaline concretes

NOS OF COMPOSITION	LATEX CONTENT, %	TIME OF APPEARENCE OF INITIAL CRACKS, hrs	COMPRESSIVE STRENGTH, MPa, SPECIMEN		COEFFCIENT OF SOFTENING
			Saturated With Water	Dry	
1	–	120	85	92	0.92
2	1.0	144	82	90	0.91
3	2.5	156	80	87	0.90

Analyzing the obtained data, presented in Table 3, suggests to conclude that the crack resristance of these concretes is greater, but the coefficient of softening is lower, when increasing the latex content to 2.5%. However, the water resistance of latex-modified slag alkaline concretes is considerably higher, it is allows to use them in water hydraulic structures.

The testing procedure for repeated shock strength was done in the latex-modified slag alkaline mortars (latex content found as 1% (in dry state) of slag mass) on.impact.testing machine with a falling load of 3 kg. The specimen were prerared in the form of cylinders of height and diameter of 25 mm. Control materials endured 15 impacts while the modified—120 impacts. This accounts for the increase in impact resistance of the samples by 8 times. Such a rise in impact resistance is explained by the peculiar structure of the modified concrete, thus stipulated by the presence of an elastic polymer layer in the contact zone of the binder and aggregate. The presence of such an elastic layer facilitates for the reduction in the internal tension of the concrete and increase of its crack resistance.

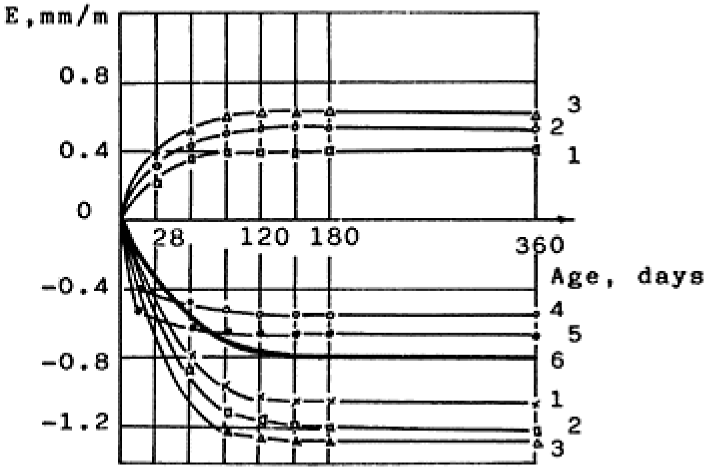

Figure 3 Kinetics of Shrinkage Deformation When Specimen are Kept in Water and Air-Dry Conditions

1–3—designations as in Fig. 1; 4 coarse aggregate concrete with 1% latex; 5—coarse aggregate concrete with 5% basalt fibers 6—fine aggrigate concrete with 2% emulsion petrolatum

CONCLUSIONS

1. The slag alkaline cements and concretes can be modified with water solution polymer additives: latex and emulsion petrolatum.
2. The slag alkaline polymer-cement concretes possess higher flexural strength, repeated shock strength, high damping properties and crack resistance.
3. The slag alkaline concretes by their physical-mechanical and special properties can be used in special fields of constraction (seismic, etc.) and non-civil engineering (vibration-damping foundations, contractions for high-way engineering, containers, foundations of precision equipment).

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RHA-CEMENT AS A REPLACEMENT FOR PORTLAND CEMENT IN RURAL TANZANIAN VILLAGES

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ABSTRACT. Rice husk ash (RHA) was emphasized as (partial) replacement of Portland Cement (PC) for application in rural areas in Tanzania. Ashes were investigated in controlled-burning conditions. Further, the effects of various additions of RHA produced in a specially designed metal kiln were studied on setting times and strength properties of mortars in which the sand to binder weight ratio was 3. Temperature and duration of incineration were varied. Composition, amorphousness, particle size distribution and specific surface area of ground ashes were investigated. Specimens of 40×40×160 mm (with RHA-lime, RHA-lime-clay, RHA-PC, or RHA-PC-clay binder), were tested according to ASTM standard C 348 and C 349 at 3 days, and 1, 4, 26 and 52 weeks. Maximum RHA content in the binder was 0.7. The best performing mortars complied with ASTM C 91 specifications for masonry. Low performance was observed for RHA-lime-based mortars with relatively high RHA contents. Clay additions had a negative effect on strength, so data are not included in this paper.

Keywords: Cement Blending, Mechanics, Mortars, Rice Husk Ash (RHA).

Associate Professor Piet Stroeven is Head of the Group on Materials Engineering at the Faculty of Civil Engineering, Delft University of Technology, The Netherlands. His main research interests focus on mechanisms and processes of damage evolution in cementitious systems. He has published widely on these subjects and serves as a member in international (ACI, RILEM) committees.

Mr Elias L Sabuni is as a surveyor employed by the Municipal Council of Arusha. In 1990/91 and 1994/95 he performed research on the use of local construction materials for low-cost housing at Rajamangala Institute of Technology in Thailand and at Delft University of Technology, respectively.

INTRODUCTION

The most widely used building material in industrialized countries is based on Portland cement as a binder. One type of Portland cement is produced in Tanzania in the more developed coastal zone, but production is inadequate to facilitate extensive rural housing development of higher quality. Moreover, cement is too expensive for most villagers, since a bag of cement would consume more than half the monthly income for the average farmer in a smaller village. Tanzania has a poorly developed infrastructure. Transport of the cement to the more remote areas in the country is expensive, time consuming and not reliable, because a considerable part of the dirt roads is inaccessible in the rainy periods. As an example, the price of the cement doubles when transported from Dar es Salaam over 750 km to Mwanza, the third largest town in the country.

Attention should therefore focus on the development of appropriate technology concepts on village level, thereby making use of local raw materials and other indigenous resources. It is known for quite some time that a highly reactive pozzolanic material can be obtained from controlled burning of rice husks (a FAO review mentions two German patents dealing with the use of RHA in concrete that were registerd in 1924). Since those days, a considerable amount of effort has been invested in this field and applications have been realized in South America as well as in Asia. Reference should be given here to the work of Mehta [1], based on fundamental insight into the chemistry involved in this materials technology. The use of RHA-cement has been promoted in the South-East Asian region by the ESCAP Regional Center for Technology Transfer through the organization of a series of workshops in 1978 and later years. A full session was devoted to RHA during the CANMET/ACI Conference in Milwaukee this year, showing the world-wide interest in this field [2].

Since rice is one of the major crops in Tanzania, the present study concentrated on production and application of rice husk ash in remote villages in the Mwanza region. Table 1 presents 1992/93 data on rice production in the relevant districts. This would provide for about 50 thousand tonnes per year of RHA. Maintaining cultural and traditional inherit age as to the building technology, a major application of a higher quality binding agent would be the production of a more durable and denser floor material. Another application would be stabilizing the mud used

Table 1. Rice production in districts around Mwanza

Mwanza Region	Rice Production [thousand tonnes]
Sengerema District	250
Kwimba District	200
Ukerewe District	230
Mwanza District	170
Geita District	100
Mangu District	50

Fig. 1. Building used by MRHP near Missungwi, Tanzania, to evaluate the durability of different types of plaster layers.

in the wattle and daub type of housing, leading to enhanced durability. A popular application is for plastering walls, with the idea of reducing erosion. But the present experiences by a local Non Government Organization (NGO), the Mwanza Rural Housing Program (MRHP), reveal some inverse proportionality between 'quality' of binder and durability. This is obvious, because the traditional wall construction (mud blocks or wattle and daub) lead to relatively large deformations which cannot be accommodated by the stiffer plaster layers, so they debond as a consequence and fall off. Fig. 1 shows a site of field testing by MRHP near Missungwi, close to Mwanza. Only in a more advanced situation, the (local) production could be perceived of building blocks made of masonry-quality of mortars, such as the ones developed in this research.

The objectives of this study were

- to analyse the Tanzanian husks for their usefulness in producing RHA;
- to investigate the 'optimum' production conditions for RHA;
- to determine the mechanical properties of RHA-based mortars;
- to design the field production in a Tanzanian rural area.

EXPERIMENTAL: CONTROLLED BURNING OF RICE HUSKS

The rice husks were received from the Arusha region in Northern Tanzania (ie the Ngarenarch rice mills). The raw material ranges in particle sizes from less than 0.038 mm to 2 mm. The chemical analysis revealed that the material contained 21.2% of silica (further, 26.2% cellulose, 21.2% hemi-cellulose, 11.6% lignin and

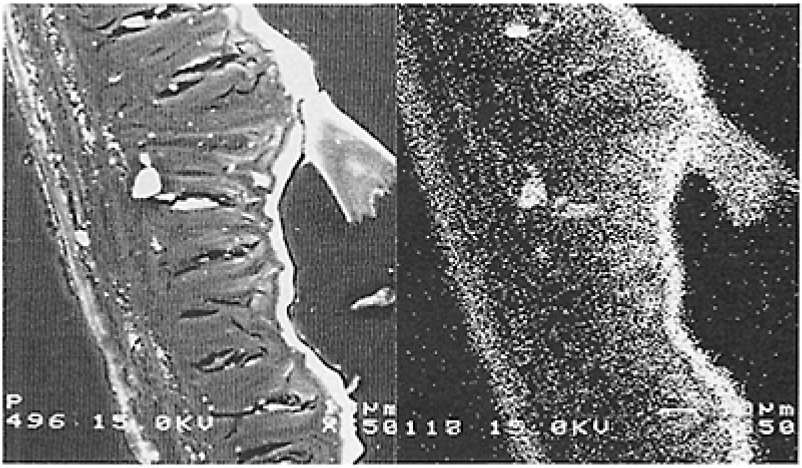

Fig. 2. BEI and X-ray image of rice husk showing cellular structure and silica distribution.

19.8% pigments, pectine and protein). A combined study using back scattered electron (BEI) and X-ray images of the husk showed the silica to be distributed mostly under the husk's outer surface, giving it an effective hardness between 5.5 and 6.5 on Mohr's scale and leading to a considerable abrasion of the grinding equipment (Fig. 2). This confirms the general concept of a soluble form of silica transported through the plant, concentrated at the outside surface of straw and husk through evaporation, whereupon it polymerizes into an opaline cellulose-silica membrane [3]. Details of the analysis are given in references [4,5].

The general composition of RHA produced from these husks was determined as an average of a series of analyses, employing an inductively coupled plasma 200 emission spectrometer (ICP-200). Table 2 shows the chemical analysis in comparison with similar data obtained on a Vietnamese rice husk type [4,5]. Based on a DTA analysis, a series of ashes was produced in a laboratory oven under

Table 2. Chemical composition of two types of rice husk ashes.

Main oxides	Vietnamese RHA [% by weight]	Tanzanian RHA [% by weight]
SiO_2	96.7	88.9
Al_2O_3	0.08	0.30

Fe_2O_3	0.03	0.19
CaO	0.30	0.43
K_2O	0.73	3.67
MgO	0.16	2.07

Table 3. Effect of temperature-time regimes on LOI of RHA.

Temperature	Duration [h]	Ash+C [%]	Colour	LOI	Ash [%]
350°C	67	24.1	grey	5.62	22.7
400°C	24	23.8	grey	6.25	22.3
500°C	20	23.5	light grey	4.86	22.4
600°C	6	32.4	blackish	27.15	23.6

Table 4. BET specific surface area of samples of RHA.

Temperature [°C]	Duration [h]	BET [m^2/g]
350	67	102
400	24	76
500	20	57
600	6	120
600	10	104
900	15	2

different temperature-time regimes, whereupon the carbon content (the loss on ignition, LOI) was determined. Some results are presented in Table 3; note that ash percentages are by weight of the husk. Another sample of husks was subjected during 10 hours to 600°C and a last one for 15 hours to 900°C, the latter ash being ‘as white as lime’. All ashes were ground for 160 minutes in an agate mortar. Thereupon, the particle size distribution (psd) was determined by Malvern 2600 equipment. Details are provided in references [4,5]. Differences between curves were not dramatic, with 50% of the particles under 5 to 10 μm. Examples of psd’s of ashes representing the 350°C–67 h and 600°C−10 h treatments are given in Fig. 3. The fineness of the ashes (ie. BET) was additionally determined by nitrogen adsorption (with Autosorb-6B gas Sorption Analyser). The data averaged over two analyses, which involve preheating at 150 and 360°C, respectively, are presented in Table 4.

An ash (from Tanzanian rice husks) produced in a kiln (also used in the present study) under conditions supposedly representative for production in the field (temperatures varying around 700°C) and intended to be used for the production of high strength concrete in Vietnam contained 23% of unburnt carbon. The ash was grinded in a 2 litres laboratory ball mill to various degrees of fineness. BET data (obtained by nitrogen adsorption) are presented for comparison in Table 5. The ash particles have a large internal porosity before grinding, as revealed by secondary electron images of the ash. The time of grinding is between brackets. Initially, the surface area increases with grinding time. But upon collapse of the porous structure, the surface area drops. The addition of a naphtalene type of superplasticizer during grinding (indicated by + in Table 5) proved to be very efficient. The best ash in Tables 3 and 4 (500°C, 20 hours burning time) is probably

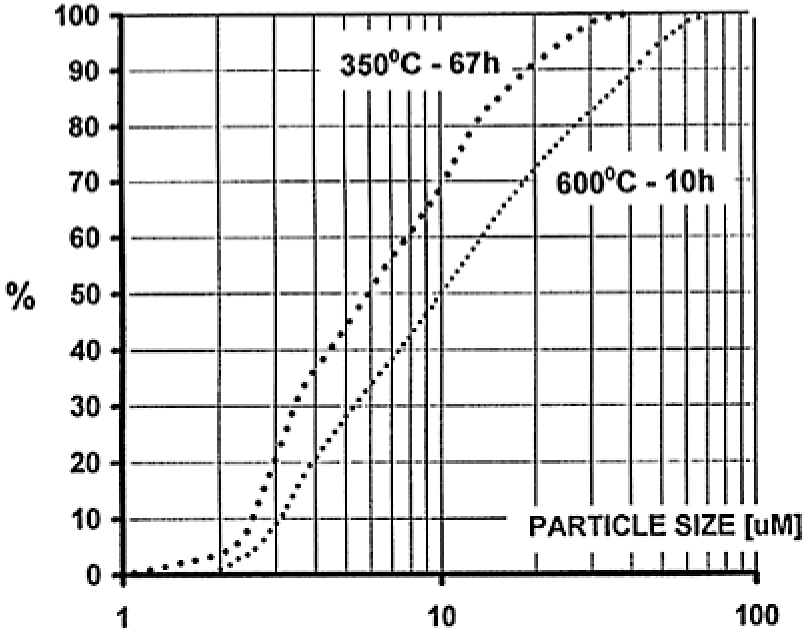

Fig. 3. Psd's of two ashes produced at 350 and 600°C, respectively, and ground for 160 minutes.

in the same state of disintegration of internal porosity. This state is to be prefered because of a reduced water demand.

The ashes were also investigated by SEM, TEM and X-ray diffraction. Up to 500°C, the secondary electron images of the ash revealed particles as spherical or globular in shape and with a porous structure. Partially crystallized ash was found at 600°C, whereas at 900°C this feature was dominant. This is confirmed by X-ray diffraction patterns. The SEM pictures revealed the globular structure to increase in size with rising combustion temperature from 5–10 μm at the lowest temperature to 10–50 μm. Individual particle and pore sizes are up to 1 μm. Agglomerates seem to become more compacted at higher combustion temperatures. Moreover, at 600°C fine porous crystalline grains, smaller than

1 μm, are displayed, possibly revealing the transformation between the amorphous and the crystalline state. This process was completed at 900°C, where a crystalline structure was observed with crystallites in the range of 1 μm or larger. Some representative samples of SEI are presented in Fig. 4.

EXPERIMENTAL: PRODUCTION OF RHA IN KILN

Stringent regulations in The Netherlands as to air pollution and safety governed the design of a kiln, intended for the production of ash under conditions representative

Table 5. BET specific surface area of RHA; between brackets is the duration of grinding [h]; only the last ash is grinded in combination with a super-plasticizer. Note that the husks are of Vietnamese origin.

RHA-quality	S_{BET} in m^2/g	ΔS_{BET} in m^2/g
RHA(0)	123	2
RHA(14)	137	11
RHA(18)	151	11
RHA(18)$^+$	58	6

of those likely to be met in rural Tanzania. A vertically-positioned cylindrical mild steel kiln provided with fire-proof lining was used. The diameter and height of the oven were 380 and 1800 mm, respectively. A longitudinal cross-section of the kiln is presented in Fig. 5. The kiln was loaded with 30 kg of husks from the top, whereupon the production could proceed continuously. Gas was used for ignition and was stopped when the temperature exceeded about 450°C. The rice husks in the top part were pre-heated by hot gasses coming from the underlying combustion chamber, where temperatures were highest. Ash was forced down by a very slowly rotating motor-driven vane and collected in the bottom part. A complete firing cycle lasted 8 hours. The ashes were left overnight at a temperature which gradually declined from 350–600°C to room temperature. Temperatures were recorded during operation at 3 positions inside the oven. For further details, see references [5,6].

A series of pilot ashes was prepared in the laboratory oven under the same temperature regimes as in the earlier test series (350, 400, 500, 600 and 900°C), but with other conditions adapted to the production process in the kiln. These ashes served as a control. The duration of the incineration process was in all cases kept at 8 hours. The ashes were ground for 1 and 2 hours, respectively. Cumulative psd and surface area by air permeability were determined. For details, see references [2,3]. Globally speaking, particles were twice as large as in the earlier series due to a shorter duration of burning and grinding; i.e. 50% of the particles was smaller than 20 to 10μm going from the ashes burnt under 350° C to those under 600 and 900°C. The specific surface area varied between 0.24 and 0.31 m^2/g. The largest value (0.31 m^2/g) was found for the 500°C, 2 hours grinding treatment, the smallest for the 900°C, 2 hours grinding treatment (0.22 m^2/g).

The moderate differences in ash characteristics made it possible to select a relatively high temperature regime for the kiln burning process. The temperature at the bottom of the combustion chamber was controlled at a level of about 600°C. Average temperatures in the combustion chamber were probably around 700°C. Unburnt parts were removed from the ash by sieving. The ash was finally ground for 130 minutes, considered to be a 'reasonable effort' under rural conditions, based on the type of applications.

BLENDING OF RHA AND PREPARATION OF MORTARS

To evaluate the practical possibilities with the RHA produced under 'field' conditions, it was used in combination with PC (RHA-PC), with PC and lime (RHA-lime-PC) and with lime (RHA-lime). Clay was sometimes added as an extra component. The blending component(s) were added in the correct proportion to the RHA which had been ground for 65 minutes. The mixture was then ground for further 65 minutes. The fineness Was checked by air permeability and found to be in the correct range of the ASTM C 595, ie. ranging from 0.27 to 0.30 m^2/g. About equal proportions of two fractions of sand were added to this binder. The size range of the sand fractions was 0.1 to 0.25 mm and 0.25 to 0.5 mm. The sand to binder ratio was 3. A minimum amount of water was added to

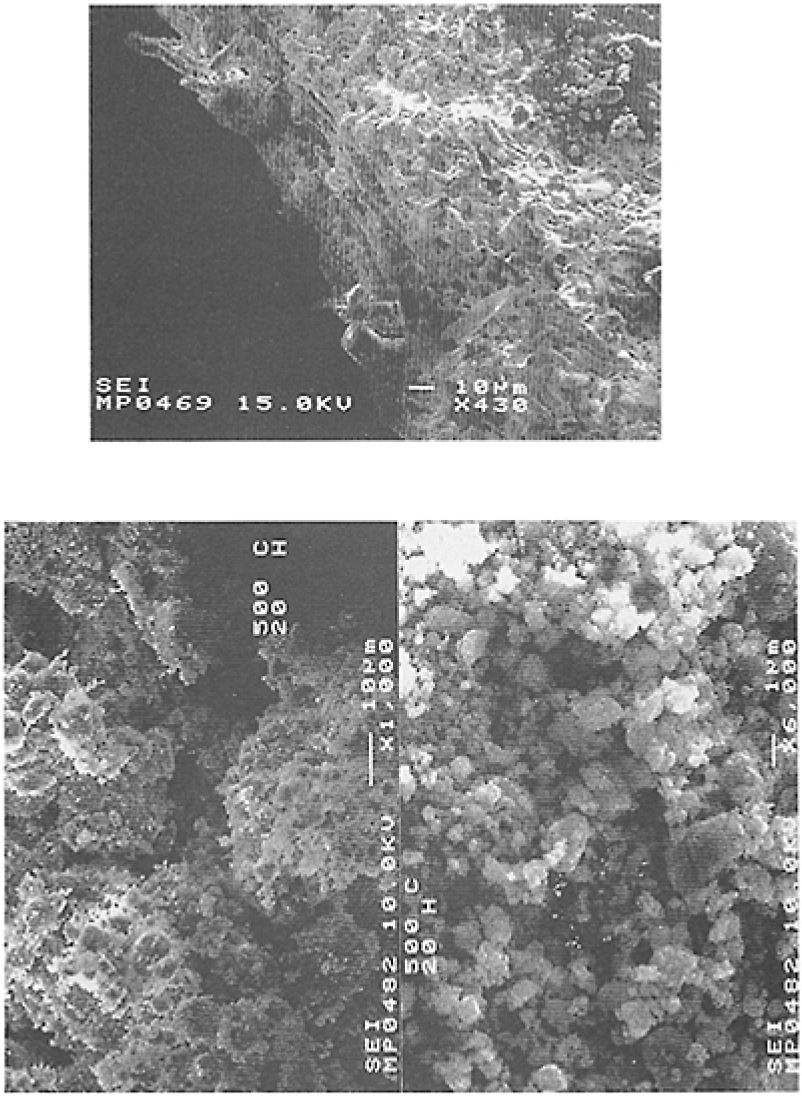

Fig. 4. SEI of husk (top), and—at two different magnifications—of ash obtained after 20 hours of burning at 500°C (bottom).

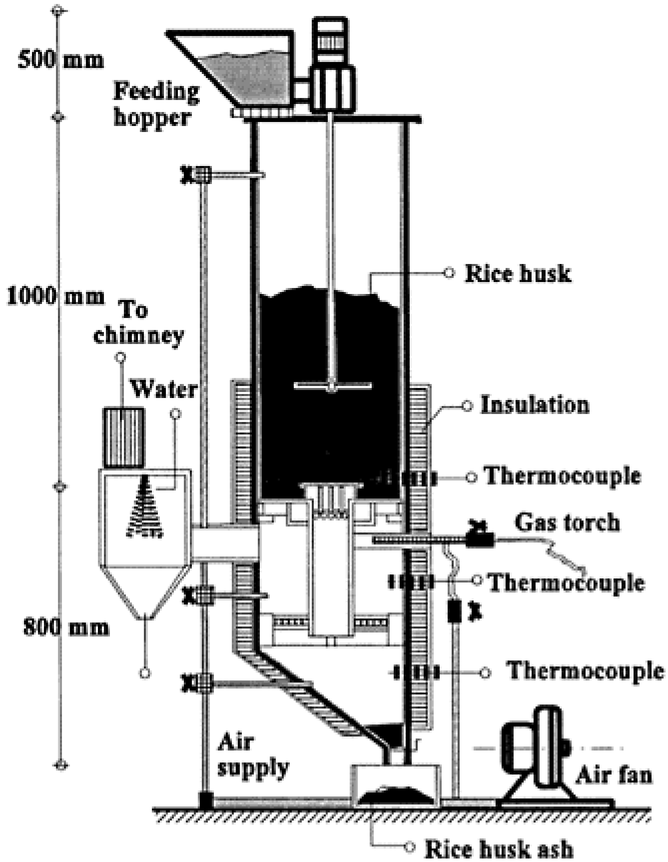

Fig. 5. Longitudinal cross-section of the kiln used for the production of rice husk ash in the laboratory.

achieve a certain level of workability, which was measured afterwards by the slump and flow test. The water to binder ratio was derived from the compositional data. As a result of this practical set up (adapting rural conditions), the slump and flow rate as well as the water to binder ratio fluctuated among 'similar' mixes of about 6 litres. Standard prisms 40×40×160 mm) were made following laboratory procedures; i.e

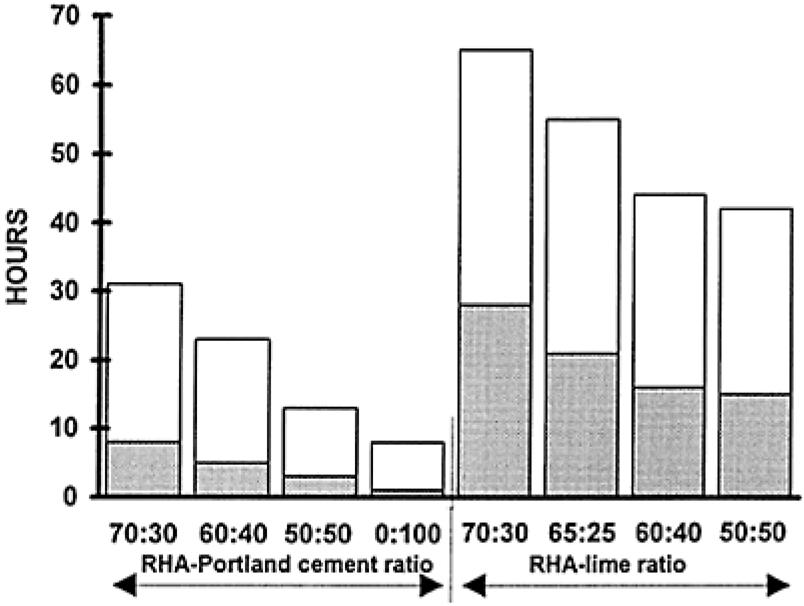

Fig. 6. Diagram showing initial (dotted bar) and final (open bar) setting times of mortars based on RHA-PC and RHA-lime binders.

moulds were filled in two separate layers, followed by 10 seconds compaction on the vibration table. The second layer exceeded the capacity of the mould. After compaction the excess of material was removed. Specimens were stored before testing in a climatized room (20°C, 99% RH). Demoulding was carried out very carefully after 48 hours.

Initial and final setting time were determined using the Vicat apparatus. Typical data of the two main RHA-blended systems are presented in Fig. 6 (open bar: final setting time; dotted bar: initial setting time). It reveals the well-known phenomenon of the RHA pozzolana delaying both setting times, in fact well above the level indicated in ASTM C 595 (for PC blended by a pozzolana). Only PC blended for a smaller part with RHA would fulfil this standard. Flow values (ASTM C 109) and slump of the various mixes slightly varied, i.e. slump between 2 and 5 mm. However, mixes had a considerable different water to binder ratio of 0.5 for the PC mortar and between 0.8 and 1.0 for the blended mixes.

TESTING AND RESULTS

Testing of mechanical properties was accomplished by means of a three-point bending test on the 40×40×160 mm prisms with a span of 107 mm (ASTM C 348). Further the compressive strength was recorded on pieces of the broken beams introducing the load via 40×40 mm platens (ASTM C 349). A total of 173 prisms was used for this field investigation. RHA content in the binder varied in all mixes between 0.5 and 0.7. This was complemented by either PC or by lime. In the PC-RHA-lime mixes the PC content was 0.1. Clay contents of 0.1 and 0.3 where applied in RHA-lime mixes. For further details, see reference [4]. Fig. 7 shows the development of bending strength within a 1 month period for the three material systems. The development of compressive strength over a period of 1 year is shown for the best performing mixes in Table 6.

Table 6. Development of compressive strength of mortars based on various binders. Note that R=RHA, l=lime, P=PC, W=Water and B=Binder. Codes are the same as in Fig. 7.

Code	Mixture Composition	W/B [g/g]	Slump [mm]	Compressive strength [MPa]				
4	0.5(R)−0.5(1)	0.92	5.0	1.30	1.72	2.55	2.29	2.48
9	0.7(R)−0.2(l)−0.1(P)	0.90	2.0	1.17	1.53	1.83	1.63	1.55
11	0.6(R)−0.3(1)−0.1(P)	0.90	2.0	1.38	2.11	2.29	2.47	2.31
12	0.5(R)−0.4(1)−0.1(P)	0.90	2.5	1.37	2.46	3.10	3.47	3.27
14	0.65(R)−0.35(P)	0.90	0.5	2.32	5.85	8.59	7.81	9.08
15	0.5(R)−0.5(P)	0.875	3.5	2.29	6.50	10.38	12.60	13.60

DISCUSSION AND CONCLUSIONS

The compressive strength of RHA-lime-PC mixes was found to fulfil the requirement of ASTM C 91 for masonry work even for a PC content under 50%. The RHA-lime mixes performed less well, except in case of mix 4 in which the RHA content was lowest (i.e 50%). For the primary purposes of densification of a floor's surface and of stabilization of the mud for walling this is sufficient.

The conditions in open heaped-up burning of the rice husk material are for most of the material probably similar to or even better than in the specially designed kiln used in the experiments [9]. The present investigations did not reveal a significant portion of the silica to be transformed into the crystalline phase for temperatures up to 700°C. Thus, open heaped-up burning would provide a cheaper solution for the primary applications of RHA in Tanzanian villages.

RHA contains only small amounts of aluminium oxide. In order to improve strength and to increase setting times, activated clay could be added. Cabrera et al [10] showed that mixes containing activated red tropical soils from the Southern Indian Shield in Sri Lanka to be superior over other mixes in which OPC was blended by PFA (containing more Al_2O_3 than the RHA). The non-activated clay used in a part of the present experiments gave rise to a strength reduction. Data are therefore not included in this paper.

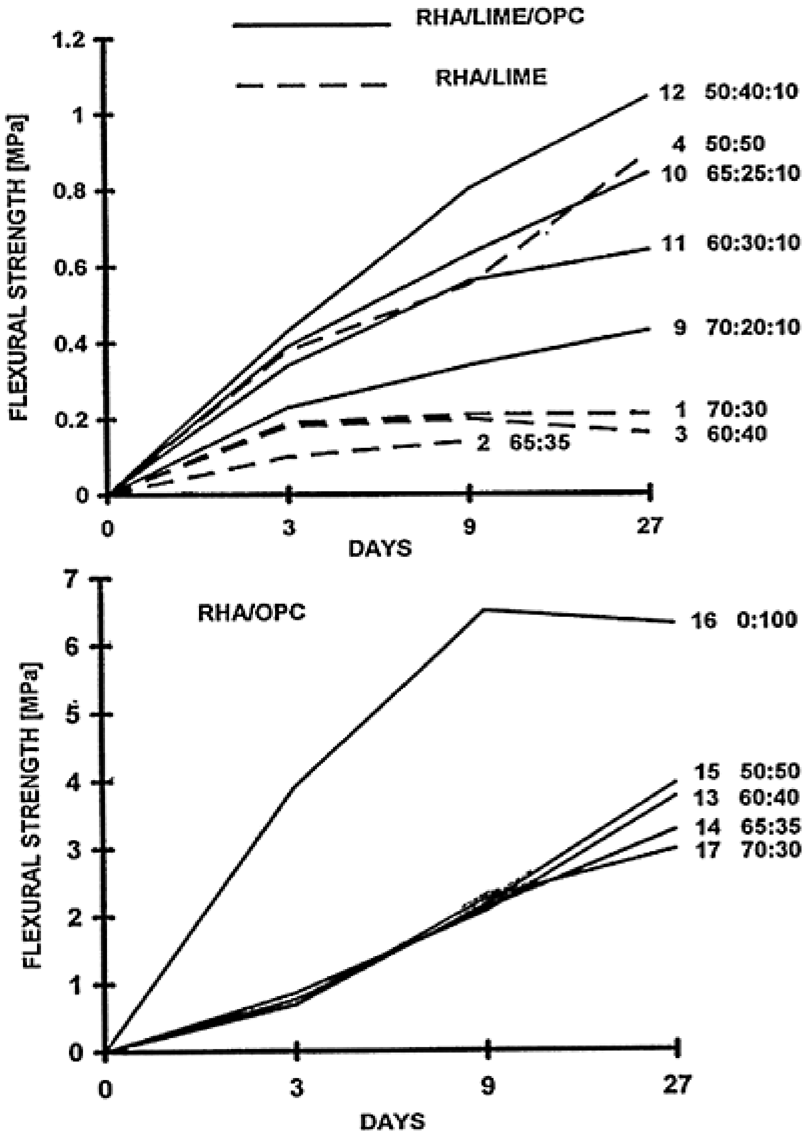

Fig. 7. 3-point bending strength of mortars based on the three respective binder systems.

Two brick-type of ovens are projected in Kalebezo village, Sengerama District, Mwanza Province, to also start the more controlled production of RHA (and clay) aiming for more advanced purposes, like the local production of bricks, tiles, etc. In some cases small additions of PC will be considered.

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EFFECT OF THE BINARY ADMIXTURES OF FLY ASH AND SHALE ASH ON THE FLUIDITY AND COMPRESSIVE STRENGTH OF CEMENT CONCRETE

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ABSTRACT. This research is intended to clarify the effect of the binary admixtures based on fly ash (FA) and shale ash (SA) on the fluidity and compressive strength of concrete. For the mixes having a water/binder ratio of 0.35, a cement content 350 kg/m^3, and binary admixtures 200 kg/m^3, the slump and compressive strength will change with FA/SA ratio, reaching the maximum at a FA/SA ratio of 4:1. On the other hand, even with the constant water/binder ratio of 0.35 and FA/SA ratio of 4.0, the cement content in the binder is very important for the slump and strength of Concrete. According to our experimental results, if we expect a high early strength for high performance concrete, the cement content should be no less than 350 kg/m^3.

Keywords: Binary Admixtures, Portland Cement(PC), Fly Ash (FA), Shale Ash (SA), High Performance Concrete(HPC), Fluidity.

Professor Feng Naiqian is Director of the Building Materials, Tsinghua University, Beijing, China. He specializes in the use of zeolite as a kind of admixture. In recent years, he has done much research on high performance concrete with the use of zeolite, fly ash, shale ash, silica fume and slag. He also specializes in fracture mechanics of concrete. He has published widely and serves on many Technical committees.

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Dr Wang dehuai is a doctor student of Prof. Feng Naiqian. He is working on the mix proportions of high performance concrete.

RAW MATERIALS

1. Fly ash: the first grade fly ash produced by Huangpu Power Station was used. Its physical properties and chemical compositions are listed in Table 1 and Table 2.

2. Shale ash: from Maoming, Guangdong Province. Its chemical compositions are listed in Table 3.

3. Coarse aggregate: crushed gravel, particle size range 5~20 mm, bulk density 1340 kg/m^3, specific gravity 2.65, silt content 0.6%, flaky and elongated content 19.7%.

4. Fine aggregate: river sand, fineness modulus 3.0, bulk density 1540 kg/m^3, specific gravity 2.6. absorption 0.5%.

5. Naphthalene superplasticizer: Brand NF.

6. Cement: #525 Portland cement.

EXPERIMENTAL DESIGNS

1. With a constant 550 kg/m^3 of binder (350 kg/m^3 of cement and 200 kg/m^3 of binary

Table 1 Physical properties of fly ash

SIEVE RESIDUE (size of 45 μm)	LOSS ONb IGNITION	WATER REQUIREMENT RATIO*	MOISTURE CONTENT	BULK DENSITY (kg/m^3)
≤12%	≤5%	≤95%	≤1%	548~563

* Assume the water requirement of mortar (cement: sand=1:2.5, by weight) with pure cement to be W_1, and that of another mortar (cement: fly ash : sand=0.7:0.3:2.5, by weight) which has the same fluidity with the former to be W_2. Then the water requirement ratio of fly ash is defined as $(W_2/W_1)\times100\%$.

Table 2 Chemical compositions of fly ash (Wt.%)

SiO_2	Al_2O_3	Fe_2O_3	MgO	CaO	SO_3	LOSS ON IGNITION
51.42	38.00	5.21	0.81	2.60	0.31	1.66

Table 3 Chemical compositions of shale ash (Wt.%)

SiO_2	Al_2O_3	Fe_2O_3	MgO	CaO	Na_2O	MgO	LOSS ON IGNITION
59.81	20.53	9.89	0.45	2.79	2.50	2.47	≤3

Table 4 Designed concrete mix proportions

SERIES No.		W/C	Water Content (kg/m^2)	CEMENTITIOUS MATERIALS (kg/m^2)			Mineral admixtures. (%)	Sand (kg/m^2)	Coarse Agg. (kg/m^2)	Super-plasticer (NF) (kg/m^2)
				PC	FA	SA				
1	1	0.35	193	350	200	0	36.4	484	1246	2.93
	2	0.35	193	350	160	40	36.4	484	1246	2.93
	3	0.35	193	350	133	67	36.4	484	1246	4.27
	4	0.35	193	350	100	100	36.4	484	1246	4.27
2	5	0.32	178	336	224	0	40	480	1250	4.48
	6	0.32	178	336	181	43	40	480	1250	4.48
	7	0.34	190	336	167	57	40	480	1250	5.04
	8	0.35	195	336	148	76	40	480	1250	5.60
	9	0.38	212	336	112	112	40	480	1250	5.60
3	10	0.31	177	314	256	0	45	470	1250	4.56
	11	0.31	177	314	204	52	45	470	1250	5.13
	12	0.31	177	314	190	66	45	470	1250	5.13
	13	0.32	183	314	171	85	45	470	1250	5.70
	14	0.37	211	314	128	128	45	470	1250	5.70
4	15	0.30	174	290	290	0	50	460	1250	4.46
	16	0.31	180	290	233	57	50	460	1250	4.46
	17	0.32	186	290	219	71	50	460	1250	5.22
	18	0.34	197	290	195	95	50	460	1250	5.22
	19	0.40	232	290	145	145	50	460	1250	5.80

admixtures), and water/binder ratio of 0.35, the varieties of compressive strength and fluidity with different FA/SA ratios were investigated.

2. With a constant 560 kg/m^3 of binder (336 kg/m^3 of cement and 224 kg/m^3 of binary admixtures), the varieties of compressive strength were investigated with different water/binder ratios (0.32, 0.34, 0.35, 0.38), so did the effect of FA/SA ratio on the compressive strength with a constant water/binder ratio of 0.32.

3. With a constant 570 kg/m^3 of binder (314 kg/m^3 of cement and 256 kg/m^3 of binary admixtures), the varieties of compressive strength were investigated with different water/binder ratios (0.31, 0.32, 0.37), so did the effect of FA/SA ratio on the compressive strength with a constant water/binder ratio of 0.31.

4. With a constant 580 kg/m^3 of binder (290 kg/m^3 of cement and 290 kg/m^3 of binary admixtures), the varieties of compressive strength were investigated with different water/binder ratios (0.30, 0.31, 0.32, 0.34, 0.40)

The correspondent concrete mix proportions are listed in Table 4.

Table 5 The experimental results

SERIES	No.	SLUMP (mm)	COMPRESSIVE STRENGTH (MPa)		
			3d	7d	28d
1	1	165	33.9	45.0	55.6
	2	150	45.8	55.7	66.5
	3	190	49,1	58.4	64.3
	4	105	54.8	60.6	64.3
2	5	175	29.1	49.2	60.3
	6	190	36.8	56.8	63.0
	7	190	24.9	53.5	58.9
	8	180	32.6	47.5	55.1
	9	180	35.6	49.7	51.3
3	10	205	31.0	39.3	61.6
	11	215	36.1	48.2	64.0
	12	175	39.1	52.9	65.2
	13	180	38.0	49.9	62.4
	14	95	33.2	44.3	53.6
4	15	205	36.1	48.1	62.4
	16	180	40.4	54.0	63.2
	17	185	39.4	55.7	60.2
	18	180	36.6	50.7	57.6
	19	195	32.9	42.6	56.0

PREPARATION OF SPECIMENS

According to each mix proportion, 0.015 m^3 of fresh concrete was mixed in a forced mixer for 3 minutes. After the measurement of slump value, it was molded with a standard vibration, and the mold was removed after 24 hours. All specimens were cured under the standard conditions for the test of compressive strength.

RESULTS AND DISCUSSIONS

All the experimental results are listed in Table 5.

1. With the same water/binder ratio of 0.35, binary admixtures content, the slump values and the compressive strengths at various FA/SA ratios at 3, 7, 28 days are shown as Series 1 in Table 5, and Figure 1 shows the relationship between compressive strength and FA/SA ratio. It shows that when the proportion between FA and SA is 4:1, the concrete strength at 28 days reaches the maximum value (66.5MPa). With the increment of SA proportion, the concrete strengths at 3, 7 days increase considerably, but that of 28 days decreases slightly (64.3MPa), while the quantity of superplasticizer used increases, and the slump value drops. Thus, considering the slump value and later strength of concrete, the optimum proportion between FA and SA is 4:1.

2. With constant total quantities of binder and mineral admixtures, the slump values and compressive strengths of concrete with different water/binder ratios are shown as Series 2 in table 5 and Figure 2. It is clear that the larger the water/binder ratio, the lower the concrete strength. In order to maintain slump values, the dosage of superplasticer must be raised with the increasing proportion of SA. Keeping the water/binder and the dosage of superplasticer constants, the slump values of concretes made with only FA admixture(224 kg/m^3) and those made with FA(181 kg/m^3) and SA (224 kg/m^3) admixture (FA:SA=4:1) are approximately the same, but the strengths of the later at 3, 7 and 28 days are higher, which is consistent with the discussion of part 1.

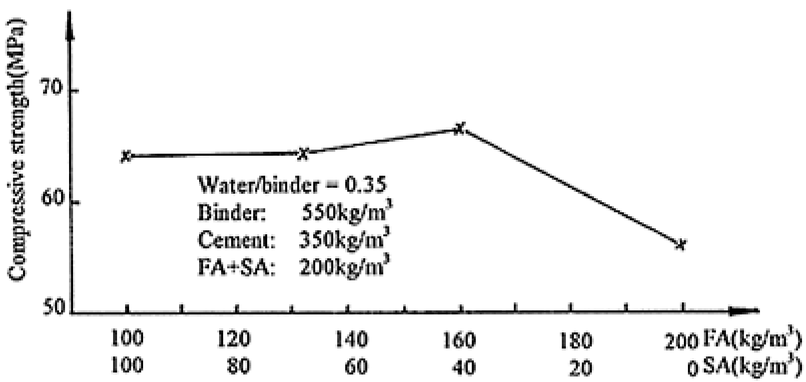

Figure 1 Effect of FA/SA ratio on compressive strength

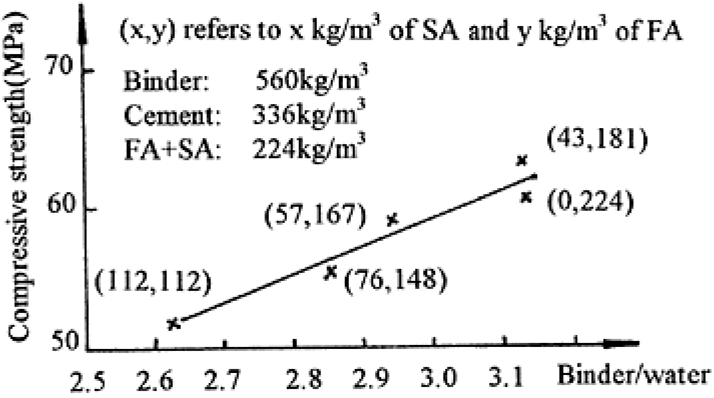

Figure 2 Effect of binder/water ratio on compressive strength

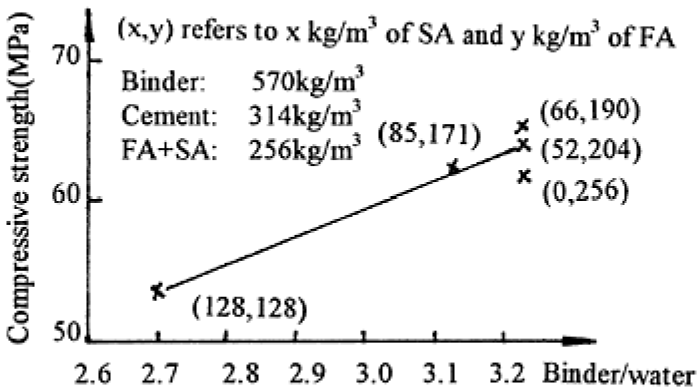

Figure 3 Effect of binder/water ratio on compressive strength

3. Keeping the binder and mineral admixtures contents the same, the strengths of concretes with different water/binder ratios and the relationships between the concrete strengths and FA, SA proportions on the same water/binder ratio are shown by Series 3 in Table 5 and Figure 3. It can be seen that: ①the larger the binder/water ratio, the higher the concrete strength. ②With the same water/binder ratio (0.31), the effect of FA admixture(256 kg/m^3) only is less distinctive than that of binary admixtures (FA:SA=4:1~3:1), which is also consistent with the discussion of part 1.

4. Keeping the contents of binder and mineral admixtures fixed as 580 kg/m^3 and 290 kg/m^3, respectively, the strengths with different water/binder ratio are shown by Series 4 in Table 5 and Figure 4.

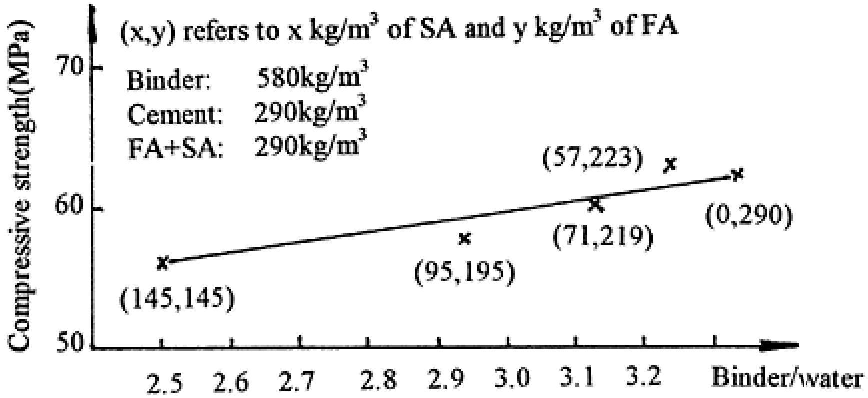

Figure 4 Effect of binder/water ratio on compressive strength

It is a general tendency that concrete strengths drop with the increment of water/binder ratio. But the proportion of FA and SA in concrete can affect its strength considerably. For example, the strengths at 3, 7, and 28 days of concrete with only FA admixture and W/C of 0.30 (No. 15) are less than those of concrete with FA-SA binary admixtures and W/C of 0.31 (No. 16). Thus, it is important to use binary admixtures and use them at proper proportions in concrete.

Furthermore, even with the same water/binder ratio, the proportion of cement is very important. For example, the cement contents of No.3 and No.8 are 350 kg/m^3 and 336 kg/m^3, the strengths at 3, 7, and 28 days of the former are 50%, 23%, and 17% higher that those of the later, respectively, but the water/binder ratios and the proportions between FA and SA are the same. Therefore, in order to gain higher early strength, the cement content cannot be too low.

CONCLUSIONS

1. High performance concrete with slump of 18–20 cm and compressive strength of more than 60Mpa can be made with a cement content of 350 kg/m^3, a mineral admixtures content of 200 kg/m^3, a water/binder ratio of 0.35, and superplasticer of 0.8% by weight of binder.

2. The strength of concrete with FA and SA binary admixtures (FA: SA=4:1~3:1) at 3, 7, and 28 days are higher than that with FA admixture only.

3. In order to produce high performance concrete with compressive strength of more than 60 MPa, the cement content should be no less than 350 kg/m^3.

A STUDY OF CARBONATION IN BINDING SYSTEM WITH CEMENT AND HYDRAULIC ADMIXTURES

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ABSTRACT. Concrete carbonation is a main factor that can lead to reinforcement corrosion due to the reduction of the intergranular solution alkalinity. The braking of the carbonation process is strongly linked to the structural compactness of the binder matrix. This paper presents the theoretical and experimental aspects concerning the carbonation of some hardening structures (pastes, mortars, concretes) made of cement with and without hydraulic admixtures. Fly ash and silica fume have been used as hydraulic admixtures that substitute cement. As part of this paper we envisaged the main influence factors regarding the composition and the curing conditions. In order to illustrate the carbonation, we have used colorimetric tests, thermal analysis and X-ray diffraction. The theoretical and practical conclusions of the study are important for the durability of reinforced concrete buildings.

Keywords: Carbonation, Fly ash, Silica fume, Ordinary Portland cement, Curing, Corrosion.

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INTRODUCTION

The utilisation of industrial wastes such as: slag, fly ash, silica fume, sludge, phosphogypsum is nowadays an economic and ecological necessity throughout the world. In Romania, a particular attention has been paid to the utilisation of fly ash and silica fume in hardening structures that make use of their pozzolanic ability in binding systems with or without cement [1]. The use of these two wastes in cement based hardening structures (pastes, mortars, concretes) requires improvements in composition and in curing, both associated with the capacity of carbonation of these structures. Therefore, this paper deals with establishing the carbonation depth for different proportions of hydraulic admixtures as cement substitute in connection with other composition and curing factors that influence the carbonation capacity. The results of the study are limited at ages of 2–3 years, but they will be completed with new data in time. Some very interesting documentary syntheses and critical appreciations have been done in Romania in the field of concrete carbonation and reinforcement corrosion [2,3], that should be completed with the long—term research of the Japanese scientists [4,5].

EXPERIMENTAL DETAILS

Materials

Ordinary Portland cement I 35 (type BS 12:1978), silico-aluminous fly ash and silica fume were used in these tests. Quartz sand in 0.08–2 mm size for mortars and natural aggregates up to 31 mm size for concretes were also used.

Mix Proportions

The proportions of hydraulic admixture (substituting cement) in pastes and mortars were of 0, 10, 20, 30, and 50%. The water/cement (W/C) or water/cement+ admixture (W/C+A) ratio was fluctuating, but the consistency was kept constant. To stand out the influence of W/C or W/C+A ratio, various types of pastes were used: normal consistency pastes and more fluid ones (+20% water). For mortars, two consistencies were also used: C_1=3 cm and C_2=8 cm. Only fly ash in the following dosages was used in the preparation of concrete specimens:

—100 kg/m^3 (10 FA1) and 180 kg/m^3 (10 FA 2) for C10, workability S_1: slump=10 mm.

—100 kg/m^3 for C15 and C20 (15 FA 1 and 20 FA 1), workability S_2: slump =55 mm. A variable quantity of mixing water ensured the same workability for control test concretes and for fly ash admixture concretes. For C15 a more fluid concrete was prepared (slump=95 mm, noted F).

Curing Environments

The specimens were preserved for different periods of time (5, 14, 21 days and one year) in moist air (95% RH), followed by curing in ordinary conditions of temperature and humidity (23°C, 50–60% RH). The concrete specimens were cured indoors and outdoors after moist air curing.

RESULTS AND DISCUSSION

The test specimens for the carbonation study were prisms of 40×40×160 mm for pastes and mortars and cubes with 100 mm side for concretes. For the reinforcement corrosion study, cubes with 100 mm side and cylindrical specimens 150 mm in diameter and 150 mm high were used (this study is not subject of the present paper). In order to appreciate the concrete degree of carbonation, several methods are quoted in literature: the colorimetric test with alcoholic solution phenolphthalein 1%, thermal analysis, X-ray diffraction, pH-method, radioactive CO_2 marking and the use of manganese hydroxide [6]. We have mainly used the colorimetric measuring the average carbonation depth (Dc) in the discoloured (neutralised) zone. The carbonation depth (higher in the upper part) was established in transversal sections obtained by splitting the specimens by the means of special designed devices. We also used the thermal analysis and X-ray diffraction on pastes and mortars with significant carbonation.

Pastes Carbonation

The results of carbonation of cement pastes with or without hydraulic admixtures are presented graphically in Figures 1a, b, c, d for normal consistency pastes. The influence of the hydraulic admixture nature is shown in Figure la (silica fume pastes—SF) and in Figure 1b (fly ash pastes—FA). Silica fume reduces more than fly ash the intergranular solution alkalinity and contributes to more compact structures that oppose to the diffusion of the carbonic gas. When using silica fume, it is necessary to homogenise it well with cement, in dry conditions, and to mix longer the paste after adding the water (because of the emphasised agglomeration tendency). Figures 1c and 1d reveal the influence of initial curing period in moist air for paste of cement with fly ash admixture; it can be seen that the carbonation depth diminishes with the increase of the duration, due to the decrease in micro-cracking trend. For fluid pastes, the average carbonation depth was higher than the one obtained for normal consistency pastes; in this case, the capillary porosity is higher and the diffusion of the carbonic gas is easier. The data obtained show clearly the influence of composition and curing factors on carbonation.

Mortars and Concretes Carbonation

The average carbonation depth for cement mortars with or without hydraulic admixture is shown in Figures 2a, b. These figures refer to 3 cm consistency mortars made of silica fume and fly ash. The test specimens were preserved in air without curing period in moist air. The 8 cm consistency fluid mortars were made using fly ash and their carbonation depth proved to be higher (full carbonation at one year). Hence, the influence of the hydraulic admixture proportion and of the consistency on carbonation is clearly reflected.

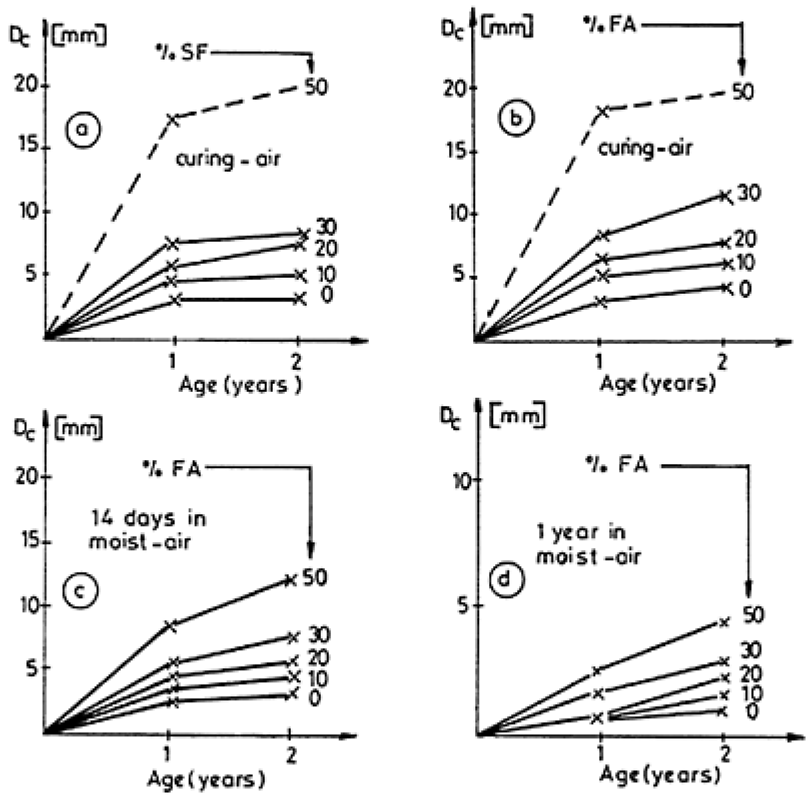

Figure 1a, b, c, d. Depth of carbonation for pastes of normal consistency.

Concretes with characteristic resistance of 10, 15 and 20 N/mm^2 were used, for which the carbonation is emphasised. The average carbonation depth for these concretes is shown in Figures 3, 4 and 5. One can see the significant influence on carbonation of the following factors: concrete resistance class, proportion of admixture (fly ash), fresh concrete workability and initial preserving period in moist air. For concrete cured indoors, the rate of carbonation was much higher than that of concrete cured outdoors because the concentration of CO_2 was higher and there was no additional supply of water. In Romania, studies on concretes with silica fume admixture up to 20% were also done.

Thermal Analysis and X-ray Diffraction

To highlight the carbonation obtained by these methods, we have used pastes and mortars at which the carbonation depth was marked and the carbonation front well delimited. Using the thermal analysis, mass losses appear due to different decarbonation between the carbonated zone and the non-carbonated one; for instance 1.68% and 0.64% for cement pastes with 50% fly ash admixture. The diffraction figures on pastes and mortars show alterations of diffraction lines between the carbonated zone and non-carbonated one. Latest data regarding 3-year carbonation indicate that the moderation trend continues in time. The evolution of carbonation

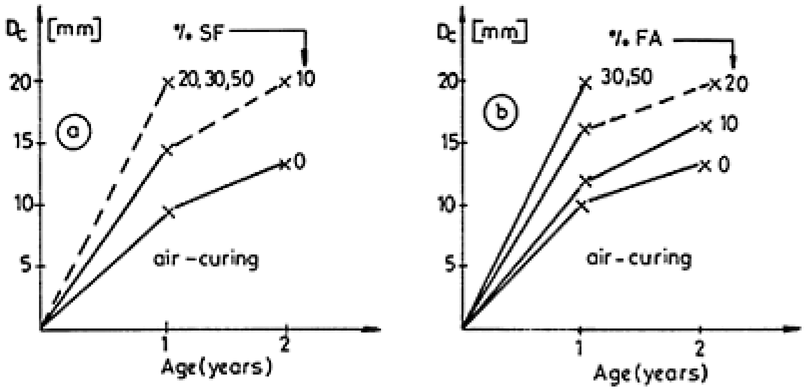

Figure 2 Depth of carbonation for mortars.

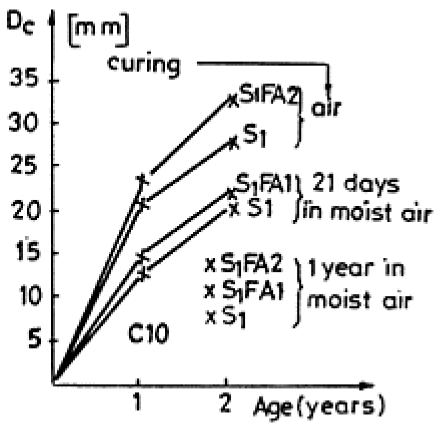

Figure 3. Depth of carbonation for concrete class 10 N/mm^2.

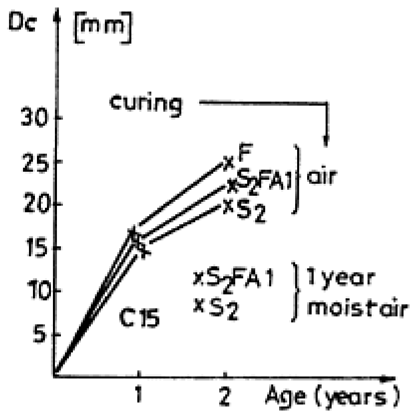

Figure 4. Depth of carbonation for concrete class 15 N/mm^2.

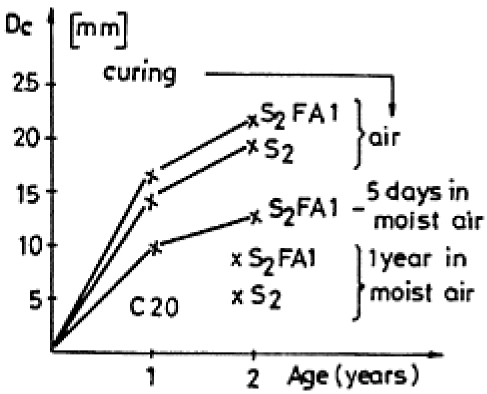

Figure 5. Depth of carbonation for concrete class 20 N/mm^2.

follows a parabola, according to the relation: $Dc = a + b\sqrt{t}$, where a, b are constants, function of the “hardening structure biography” and *t* is the age.

CONCLUSIONS

1. Natural carbonation occurs at all cement based hardening structures with or without hydraulic admixtures. The average carbonation depth reaches maximum values for air-curing (with no initial hardening period in moist air) and minimum values after long

preserving in moist air. An important role has the W/C or W/C+A ratio which influences the porosity and the carbonic gas diffusion.

2. At the hardening structures with hydraulic admixtures, the carbonation depth rises along with the proportion of cement substitute admixture; up to 10–20% of hydraulic admixture, the values of the carbonation depth do not differ too much of those of the control tests.

3. The colorimetric test used to reflect the carbonation proved to be convenient and fast. Thermal analysis and X-ray diffraction provide differentiations between the carbonated layer and the non-carbonated one, but they are difficult to exploit in practice.

4. An initial curing period of 7–14 days in moist air would eliminate the risk of steel depassivation through carbonation for reinforcements with minimum 20 cm cover. When the carbonation reaches the steel corrosion becomes possible.

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APPLICATION OF THE CEMENT HYDRATION EQUATION TO CONCRETE

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ABSTRACT. The cement hydration equation is applied to determine the equation of the compressive strength of concrete. The equations of the compressive strength of two concrete mixtures with different values of cement content were defined. Based on these equations the compressive strength of the two concrete mixtures was estimated for the ages of 3, 7, 14, 28, 50, 90, 180, 365, 730 and 5475 days. For the estimation of the best value of the hydration number p, mortar mixture was prepared according to DIN 1164. Based on this mixture’s compressive strength measured at several ages, the best value of p was estimated with the help of an appropriate P/C programm.

Keywords: Cement hydration equation, Hydration criteria, Hydration number, Linear regression, Equation of the compressive strength of concrete, Pozzolan cements.

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INTRODUCTION

Since strength at age of 28 days is regarded as a criterion of desirable quality and design, the estimation of the strength of concrete at later ages is not usually accomplished, because long time studies, up to the end of the hydration, are needed. Such studies are not available in the literature. The solution in this problem is given by the cement hydration equation [1–4]. Using this equation it is possible to predict the compressive strength of concrete after the age of 28 days, without performing compressive tests.

The Cement Hydration Equation introduced by Prof. K.Sideris [1–4] is expressed by $K=K_{\infty}\pm b*t^{-P}$, where K represents a hydration criterion, K_{∞} and b are constants, t is the hydration time and p is the hydration coefficient. As hydration criteria can be considered the heat of hydration, the non-vaporable water, the $Ca(OH)_2$ elimination, the density, the total porosity, the water sorption or the compressive strength (of cement paste, mortar or concrete).

The Cement Hydration Equation was applied to the compressive strength of concrete which was determined for a hydration time up to 90 days [5,6,7]. An example was given for the values of the compressive strength of concrete according to F.M.Lea [8] as it is shown in Figure 1.

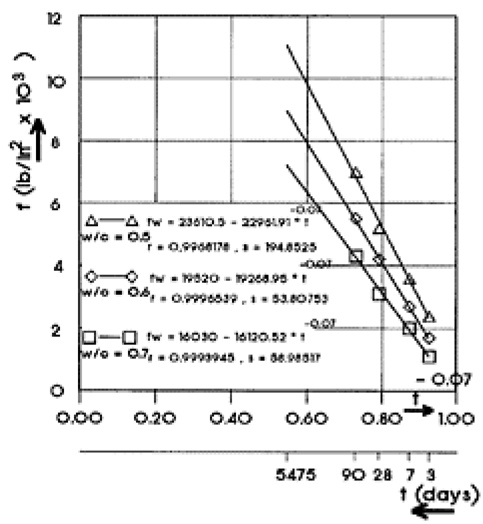

Figure 1 Comressive strength of concrete versus modified time scale.

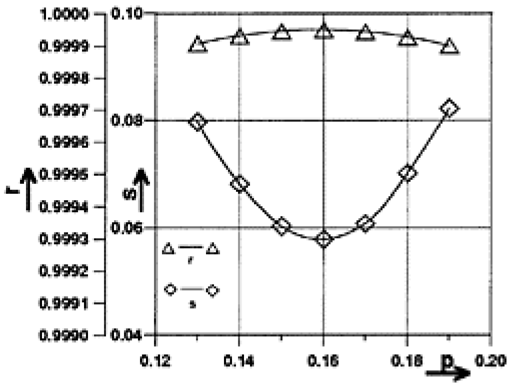

Figure 2 Estimation of the best value of p.

In the present work the Cement Hydration Equation is applied in order to predict the compressive strengths of two concrete mixtures with different cement content, up to the age of 5475 days. The hydration number p is determined from the mortar compressive strength according to DIN 1164 for hydration times 3, 5, 7, 10, 14, 18 22 and 28 days, according to [1], for more than 3 measured values of the compressive strength.

CEMENT HYDRATION EQUATION

The first and second phases of the hydration [1–4], characterized by a hydration criterion K in the co-ordinate system with K on the ordinate and $(1/t)^p$ on the abcissa, follow a straight line path. The quantitative relationship is given by the following equation: $K=K_\infty \pm b^*t^{-p}$ (1) Equation (1) corresponds to the exponential function of the general form $y=a \pm b^*t^{-p}$ in which:

K: is the hydration criterion (heat of hydration, non-vaporable water,

porosity, density, sorption, Ca(OH)2 elimination, compressive strengths of paste, mortar and concrete, etc.)

K_∞: is a constant, which represents the intersection of the straight line of equation (1) with the ordinate

b: is the slope of the straight line of equation (1).

t: is the hydration time expressed in days (t>>0).

p: is the hydration number, depending only on the chemical composition of the cement type used.

The cement hydration equation (1) has been applied to a number of criteria indicated below:

Table 1. Hydration criteria.

1.	Heat of Hydration (cal/gr)	Q	$Q=Q_{\infty}-bt^{-P}$	[1–4]
2.	Non-vaporable water (%)	W_a	$W_a=W_{n\infty}-t^{-P}$	[1–4]
3.	$Ca(OH)_2$ elimination (%)	CaO	$CaO=CaO_{\infty}-bt^{-P}$	[1–4]
4.	Density (gr/cm^2)	d	$d=d_{\infty}+bt^{-P}$	[1–4]
5.	Porosity (%)	P	$P=P_{\infty}+bt^{-P}$	[1–4]
6.	Specific surface	O_S	$O_S=O_{S\infty}-bt^{-P}$	[1–4]
7.	Water sorption	W_a	$W_a=W_{a\infty}+bt^{-P}$	[6]
8.	Compressive strength of mortar	β_P	$\beta_P=\beta_{P\infty}-bt^{-P}$	[1–4]
9.	Compressive strength of cement paste	β_d	$\beta_d=\beta_{d\infty}-bt^{-P}$	[1–4]
10.	Compressive strength of concrete	f_W	$f_W=f_{W\infty}-bt^{-P}$	[1–5, 7, 9, 11]
11.	Pulse velocity (Km/sec)	V_t	$V_t=V_{\infty}-bt^{-P}$	[9]
12.	Rebound hammer units	R_t	$R_t=R_{\infty}-bt^{-P}$	[10]

Equation $K=K_{\infty}\pm b\,t^{-P}$ applies to the hydration criteria shown in table 1, only if the type of the cement used remains the same (Portland cement, pozzolanic cement, p.f.a. cement and slag-rich blast furnace cement) [1–4]. It is important to notice that the value of the hydration number p remains the same as long as the chemical composition of the cement type used also remains the same.

The value of the hydration number p does not depend on:

1. the hydration criterion used
2. the curing temperature [7–60°]
3. the fineness of the cement
4. the ratio w/c
5. the aggregates' grading curve and the aggregates' quality when the hydration criterion of the compressive strength of mortar or concrete is examined.

The value of the hydration number p can be estimated with the use of any hydration criterion shown in table 1. The accuracy of the determination of the value of p depends on the accuracy of the measured values of the hydration criterion used.

Evaluation of the hydration number p.

For the evaluation of the hydration number p of a particular cement with known chemical composition, two different methods may be used:

Method 1: The evaluation of the hydration number p may be obtained using the formula:

$$\frac{(1/t_1)^p-(1/t_2)^p}{(1/t_1)^p-(1/t_3)^p}=\frac{K_2-K_1}{K_3-K_1}$$

In this case three measured values (K1, K2, K3) of the hydration criterion used are needed. These values must be determined with high accuracy in order to obtain a reliable value of the hydration number p. Even a small deviation of one or more of the measured values leads to unreliable values of p. Such accuracy of the measured values K1, K2 and K3 may be obtained only using the hydration criterion of the heat of hydration. The use of the formula shown above is not recommended when the hydration criterion of the compressive strength of cement paste, mortar or concrete is used. In this case the second method mentioned below is suggested.

Method 2: This method can be used with any of the hydration criteria shown in table 1. It is specially recommended though for the evaluation of the compressive strength, since the measured values in this case are not of high accuracy. In this case the use of 6–8 measured values is recommended. The value of p is estimated with higher accuracy if the criterion of the compressive strength of mortar is used, according to DIN 1164 or ASTM/C-349.

EVALUATION OF THE BEST VALUE OF THE HYDRATION NUMBER P

In order to evaluate the value of the hydration number p, the compressive strength of a mortar mixture (M.M. 1:3:0.5) was experimentally determined for the ages of 1, 2, 3, 5, 7, 10, 14, 18, 22 and 28 days. Specimens of dimensions 4×4×16 cm were prepared according to DIN 1164.

Use of different values of p.

Initially, the point of intersection of the hydration lines for the two phases is determined approximately by graphical means (Fig. 3). This is accomplished by substituting different p values, until the measured values lie on two straight lines. The best value of p is determined based on the measured values of the criterion β_D for the ages of 3, 5, 7, 10, 14, 18, 22 and 28 days for different values of p (p=0.13, 0.14, 0.15, 0.16, 0.17, 0.18, 0.19). The compensating computation (linear regression) of the β_D values versus the corresponding ages, expressed by the formula $(l/t)^{-p}$, leads to different values of the correlation coefficient (r) and the standard deviation (s). The best value of the hydration number, in the examined case, was estimated to be p=0.16 as shown in figure 2.

Use of special P/C programm.

The best value of the hydration number p is automatically estimated by the use of a special P/C programm. Linear regression is performed for the measured values β_D versus the corresponding hydration age (t), initially for the 6 last pairs up to the 10 pairs, as it is shown in table 3. It is also noted that the best value of p corresponds to the maximum value of the correlation coefficient (r) and the minimum value of the standard deviation (s). This value is estimated to be p=0.159≈0.16. The pairs β_{Dmeas}–t_{2-28} and β_{Dmeas}–t_{1-28} lead to unsuitable p values (p=0.001) since they belong to the first phase of the hydration.

Table 2 Use of special P/C programm.

Age (days)	**1**	**2**	**3**	**5**	**7**	**10**	**14**	**18**	**22**	**28**
β_{Dmeas} (N/mm^2)	**26.1**	**28.0**	**29.1**	**33.2**	**35.7**	**38.3**	**40.6**	**42.2**	**43.3**	**44.9**
Linear regression	–	–	–	–	7	10	14	18	22	28
for the value pairs	–	–	–	–	35.7	38.3	40.6	42.2	43.3	44.9
β_{Dmeas}–t^{-p}			p=0.183,		r=0.999838,		s=0.068144			
Linear regression	–	–	–	5	7	10	14	18	22	28
for the value pairs	–	–	–	33.2	35.7	38.3	40.6	42.2	43.3	44.9
β_{Dmeas}–t^{-p}			p=0.167,		r=0.9999099,		s=0.062434			
Linear regression	–	–	3	5	7	10	14	18	22	28
for the value pairs	–	–	29.1	33.2	35.7	38.3	40.6	42.2	43.3	44.9
β_{Dmeas}–t^{-p}			p=0.159,		r=0.999951,		s=0.057869			
Linear regression	–	2	3	5	7	10	14	18	22	28
for the value pairs	–	28.0	29.1	33.2	35.7	38.3	40.6	42.2	43.3	44.9
β_{Dmeas}–t^{-p}			p=0.001,		r=0.997450,		s=0.469371			
Linear regression	1	2	3	5	7	10	14	18	22	28
for the value pairs	26.1	28.0	29.1	33.2	35.7	38.3	40.6	42.2	43.3	44.9
β_{Dmeas}–t^{-p}			p=0.001,		r=0.989928,		s=1.019341			

COMPRESSIVE STRENGTH OF CONCRETE

Pozzolan cement was used, up to 35% content in pozzolan per weight, with a fineness of 3900 cm^2/g Blaine and compressive strength equal to 45.2 N/mm^2, as specified in DIN 1164. Limestone aggregates with a maximum

Table 3 Determination of equations of compressive strength of concrete.

Cement content 275 Kg/m^3										
Age (days)	**3**	**7**	**14**	**28**	**50**	**90**	**180**	**365**	**730**	**5475**
Measured values of f_W (N/mm^2)	11.9	18.4	24.0	28.7	31.5	35.3	38.5	41.8	44.4	–
Linear regression for the pairs f_{wMeas}–$(1/t)^{0.16}$	$fw\infty$=67.61188, b=66.69468, r=0.9997398, s=0.2653123 Compressive strength equation: $fw=67.61188–66.694688*(1/t)^{0.16}$ (N/mm^2), t≥3									

$f_{WRegression}$	***11.7***	***18.7***	***23.9***	***28.5***	***31.9***	***35.2***	***38.6***	***41.7***	***44.4***	***50.8***
Cement content 300 Kg/m^3										
Age (days)	**3**	**7**	**14**	**28**	**50**	**90**	**180**	**365**	**730**	**5475**
Measured values of f_w (N/mm^2)	16.8	23.8	28.3	32.9	37.1	39.6	42.7	46.4	48.6	–
Linear regression for the pairs $f_{wMeas}-(1/t)^{0.16}$	fw∞=71.4227, b=65.2165, r=0.9995724, s=0.3325698 Compressive strength equation: $fw=71.4227-65.2165*(1/t)^{0.16}$, (N/mm^2), t≥3									
$f_{WRegression}$	***16.7***	***23.6***	***28.7***	***33.2***	***36.5***	***39.7***	***43.0***	***46.1***	***48.7***	***54.9***

particle size of 32 mm were used. For the aggregate grading curve (B32) 8 fractions (32/16, 16/8, 8/4, 4/2, 2/1, 1/0.5, 0.5/0.25, 0.25/0) were used as specified in DIN 1025. The cement content of concrete was 275 Kg per m^3 for the first mixture and 300 Kg per m^3 for the second mixture. The water to cement ratio was equal to 0.60 for each case. The final compressive strength was calculated as the mean strength of the experimentally measured values of three specimens. Results are shown in table 3.

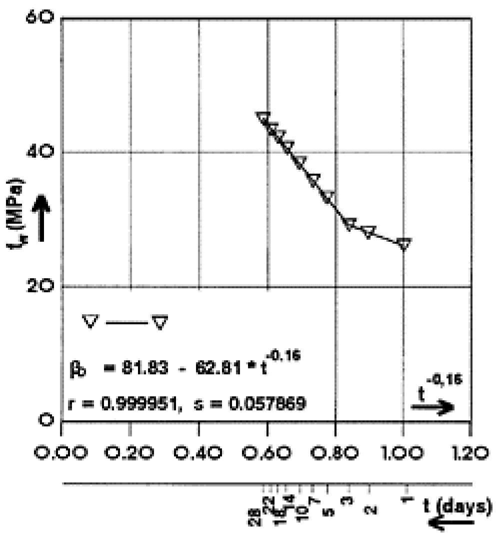

Figure 3 Compressive strength of mortar versus modified time scale.

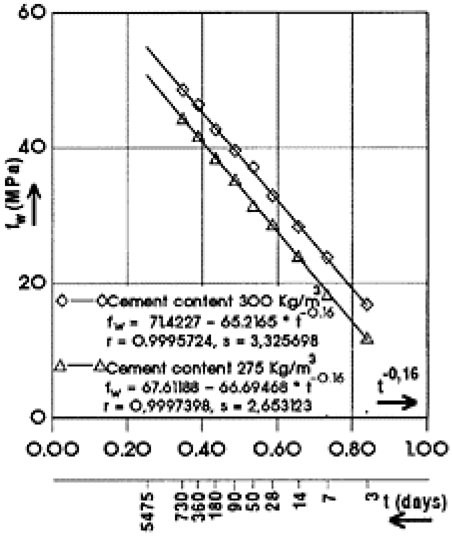

Figure 4 Compressive strength of concretes versus modified time scale.

DETERMINATION OF EQUATIONS OF COMPRESSIVE STRENGTH OF CONCRETE

Based on the values of table 2 and the hydration number p=0.16 a linear regression analysis of the compressive strength values f_w in respect to the corresponding hydration time, expressed by $(1/t)^{-0.16}$, was performed for the cases of 3, 5, 7, 14, 28, 50, 90, 180, 365 and 730 days.

Using the results of this linear regression, the equation of the compressive strength for each of the concrete mixtures was defined. Linear regression results as well as the compressive strength equations are presented in Table 3. The variation of the compressive strength versus time is presented in figure 4.

CONCLUSIONS

The equations which correlate the compressive strength of concrete mixtures with time, can be formulated by the application of the cement hydration equation. Using these equations, the compressive strength of concrete can be reliably estimated at any age, up to the end of the hydration period, which is considered to be in 15 years (5475 days) [1–4], without performing laboratory tests for long time periods. The formulation of these equations can easily be obtained using values of the concrete compressive strength, measured at the early ages of the hydration [1, pp 344, chapter 7].

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PROPERTIES OF HIGH TEMPERATURE-HUMIDITY CURED OPC-GGBS FIBRE CONCRETE ROOFING TILES

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Appropriate Concrete Technology. Edited by R K Dhir and M J McCarthy. Published in 1996 by E & FN Spon, 2–6 Boundary Row, London SE1 8HN, UK. ISBN 0 419 21470 4.

ABSTRACT. The use of ground granulated blast-furnace slag (GGBS) as a separate cementitious material added along with ordinary Portland cement in the production of fibre concrete roofing tiles has been studied.

A simple technology used in the production of the tiles which utilizes the much available sun energy to provide suitable curing temperature is described. The effects of the addition of super-plasticizing admixtures (Conplast P509, P211 and P430) on the durability is presented.

Standard testing methods for strength has also been investigated and alternative methods proposed which takes account of the profile and irregular cross-sectional properties of the Roman II tiles.

Test results indicate that better tiles can be obtained using this method of production with partial replacement of cement with GGBS.

Keywords: Roofing-Tiles, Concrete, Ground granulated blast-furnace slag (GGBS), Fibre, Roman II.

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INTRODUCTION

Due to the escalating cost of affordable housing in Zimbabwe in recent years, it has become necessary to seek ways of reducing the cost of housing. Building materials constitute a major input as they sometimes account for as much as 75% of the cost of low-income houses.

Large quantities of industrial wastes are being generated in the steel and chemical industries. The disposal of these wastes poses problems, it is cheaper to find alternative use for the wastes. The ZISCO-STEEL company in Zimbabwe produces granulated blast-furnace slag at the rate of 500 kg per tonne of hot metal. This gives about 340,000.00 tonnes of slag annually. Hence the slag mountains adjacent to the steel works.

The present work is part of a program, at the University of Zimbabwe, designed to study and to promote the use of industrial wastes in construction. The use of GGBS as a partial cement replacement material in the production of fibre reinforced concrete Roman II type of tiles have been studied and presented. The methods of testing the tiles have also been studied and further suggestions are presented.

Zimtiles and Parry Instamac are among the largest tile producers in Zimbabwe. The tiles used in this work were produced at the Parry Instamac works in Harare. The main objective of this investigation is to provide information for the use GGBS in the production of roofing tiles locally.

THE ROMAN II ROOFING TILES

Materials

The materials used in producing the tiles are Portland cement, sand, water and fibre. The basic mix is 1:3 cement: sand with the addition of 1% fibre. The most commonly used fibres are sisal, jute, henequen or coconut coir. The fibres should be flexible and not brittle and should not contain oily substances or sugar. The sisal fibre used as reinforcement in the tiles originate from sisal plants which are grown extensively in Zimbabwe. Sisal is reported to be one of the strongest natural fibres available. A report by Aziz (4) show that sisal fibres have an average diameter of 0,3 mm and a tensile strength of between 280 and 560 MPa. The fibre length is in the range 50 mm to 150 mm. The fibres were obtained by scrapping the sisal leaves, drying and chopping them ready for use. Gram (5) has shown that the durability problems in the use of sisal fibre reinforcement in tiles are due to alkali attack of the fibres by the pore water present in the cement matrix. This can be minimized if the alkalinity of the pore water is reduced by the addition of pozzolanic materials such as GGBS.

The GGBS used in the investigations originated from ZISCO-STEEL plants in Redcliff and were marketed by FOSROC. The chemical composition is shown in Table 1.

Table 1. Chemical Composition of the GGBS

Component	CaO	SiO_2	Al_2O_3	MgO	S	Na	Mn
%	36.4	30.4	16.9	11.5	2.0	0.78	0.6
Component	FeO	K	Pb	Mo	P	B	Zn
%	0.5	0.22	0.02	0.02	0.012	0.011	0.003

The Parry Associates Roman II tile is sketched in Figure 1 and is also shown on a completed roof structure in Figure 2. The tiles have a large overlap, deep valley and close fit to ensure that the roofs are leak proof without the need for underfelt or plastic sheeting. They are normally available in three thicknesses; 6 mm, 8 mm and 10 mm.

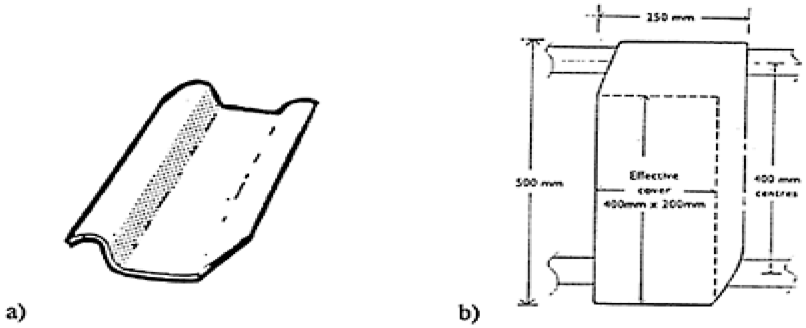

Figure 1. The Parry Roman II tile, a) In 3-dimensions b) Dimensions in plan

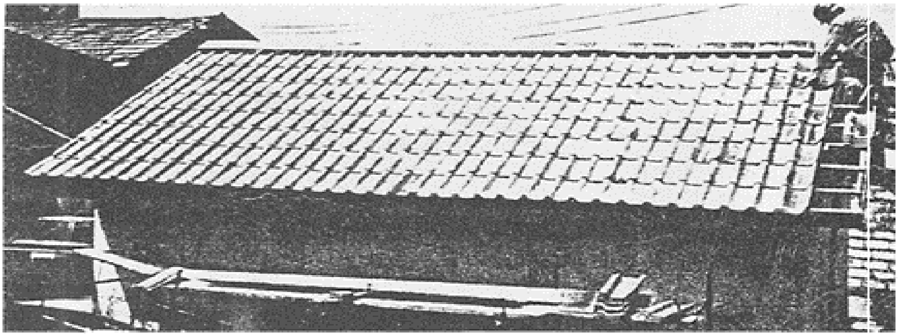

Figure 2. Roofing System using the tiles.

Production

The main equipment for making the tiles is a vibrating screeding machine powered by a multivibe vibrator on which a shaped flat screed is produced, and the moulds which give the screed its 3-dimensional tile form as shown in Figure la. The basic dimensions are shown in Figure 1b. The production technique is described in the Parry Instamac manual (1).

Series of tiles were made by replacing part of the cement in the control by GGBS in varying proportions. Samples containing the following percentages of GGBS in the binder were made: 0%, 10%, 20%, 30%, 40%, 50%, 60% and 80%.

A low cost curing technique was used which involves the use of a sealed tank covered with polythene sheet and exposed to direct sunlight. Figure 3 shows a cross section of the tank. The water level in the tank was just below the surface of the gravel such that the tiles were not in direct contact with static water. Static water tends to leave map traces on the tiles.

The heat inside the sealed tank causes vapour to rise thereby providing moisture for curing. A high temperature and humidity condition therefore prevails within the tank. The probe of a digital thermometer was inserted in the tank before it was sealed.

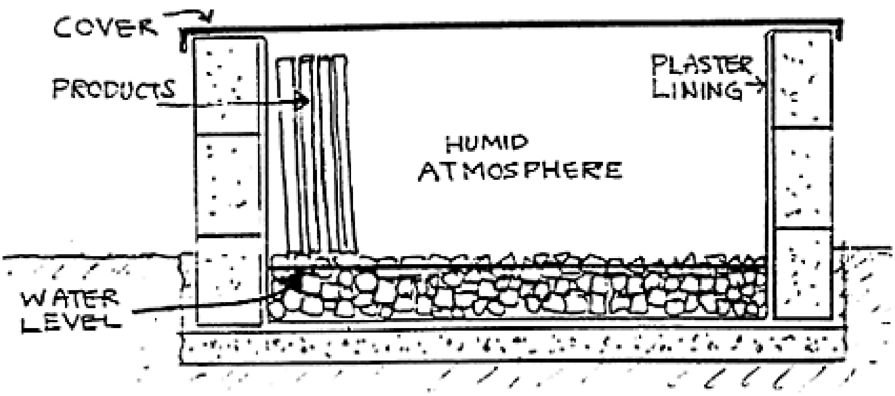

Figure 3. Sketch of the Cross Section of the Curing Tank.

An average temperature of 28°C was obtained in the curing tank over the seven-day curing period (Sept. 20—Sept. 26, 1994) compared with the average ambient temperature of 22°C. The registered day temperatures were up to 60% higher than the ambient temperature. The difference between the night temperatures was much lower Figures 4 and 5 show the temperature readings for 1 day and 7 days respectively.

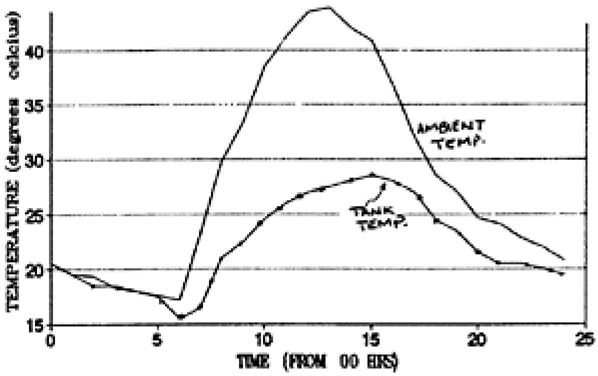

Figure 4. 24-hr. Temperature inside reading inside and outside curing tank.

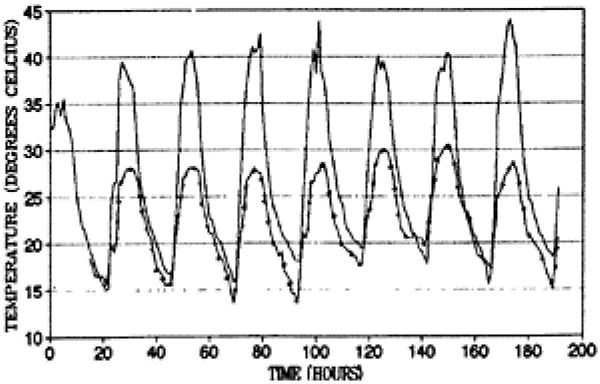

Figure 5. Temperature readings over a one-week period.

The humidities both inside and outside the curing tank were measured with wet and dry bulb hygrometers and a hygrometric table used to compute the relative humidities. The average relative humidity in the tank was 90% compared with an average of 38% humidity for the surrounding area.

The tiles were demoulded after 24 hours and then placed in the curing tank for 7 days. Previous tests have shown that curing for longer periods did not provide significant additional benefit.

After removal from the curing tank, the tiles were left to cure in air for a further 7 days before they were ready for use. The same standard procedure was followed for making all the samples.

EXPERIMENTAL

Water Permeability

Water permeability tests were carried out to the procedur described by Johansen (6). A similar method is also used at the Parry Instamac production yard for testing tiles. Four tiles were selected from each sample for the test. Two small weirs were formed on each tile and water was allowed to stand in a pool for a period of 24 hours. A tile was considered to have failed the test if after the 24-hour period there was free water on the underside. The results of the water permeability tests are shown in Table 2.

Table 2. Results of the permeability tests

SAMPLE	GGBS REPLACEMENT, %	TYPE OF ADMIXTURE	RESULT
C-0	0	No admixture	Pass
C-1	10	No admixture	Pass
C-2	20	No admixture	Pass
C-3	30	No admixture	Pass
C-4	40	No admixture	Pass
C-5	50	No admixture	Pass
C-6	60	No admixture	Pass
C-7	70	No admixture	Pass
C-8	80	No admixture	Fail
2–0	0	Conplast P211	Pass
2–1	10	Conplast P211	Pass
2–2	20	Conplast P211	Pass
2–3	30	Conplast P211	Pass
2–4	40	Conplast P211	Pass
2–5	50	Conplast P211	Pass
2–6	60	Conplast P211	Pass
2–7	70	Conplast P211	Fail
2–8	80	Conplast P211	Fail
4–0	0	Conplast P430	Pass
4–1	10	Conplast P430	Pass
4–2	20	Conplast P430	Pass
4–3	30	Conplast P430	Fail
4–4	40	Conplast P430	Fail

4–5	50	Conplast P430	Fail
4–6	60	Conplast P430	Fail
4–7	70	Conplast P430	Fail
4–8	80	Conplast P430	Fail
5–0	0	Conplast P509	Pass
5–1	10	Conplast P509	Pass
5–2	20	Conplast P509	Fail
5–3	30	Conplast P509	Fail
5–4	40	Conplast P509	Fail
5–5	50	Conplast P509	Fail
5–6	60	Conplast P509	Fail
5–7	70	Conplast P509	Fail
5–8	80	Conplast P509	Fail

Strength Tests

Tests for the strength in bending were carried out using three methods:

i) The method described in BS 550 part 2:1971;
ii) The method described in the ITW/Parry manual;
iii) A modification of the ITW/Parry method.

The BS method recommends that six tiles be selected for testing from each sample. The tiles were immersed in water for 24 hours and tested soon after removal from water. The tile to be tested was supported on two bearers. The distance between the bearers being two thirds of the length of the tile. The load is applied centrally through a third bearer of diameter 38mm. A sketch of the arrangement is shown in Figure 6. The loading was applied at the rate of 800 N/minute.

The ITW/Parry method recommends that the load be applied evenly across the center of the tile as sketched in Figure 7. The wooden platen of 25mm width is designed to follow the profile of the top surface of the tile as shown in Figure 7.

A third testing arrangement was devised in which the supports and the bearer for load application were designed to follow the surface profile of the tile supported. The arrangement is sketched in Figure 8.

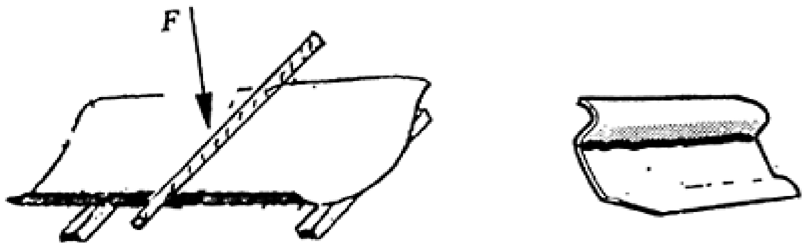

Figure 6. Sketch of the testing arrangement to BS 550 and the direction of crack propagation

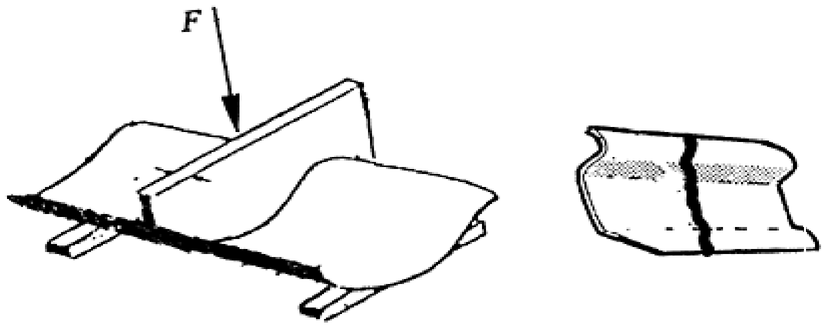

Figure 7. Test arrangement using the Parry method and the direction of crackpropagation.

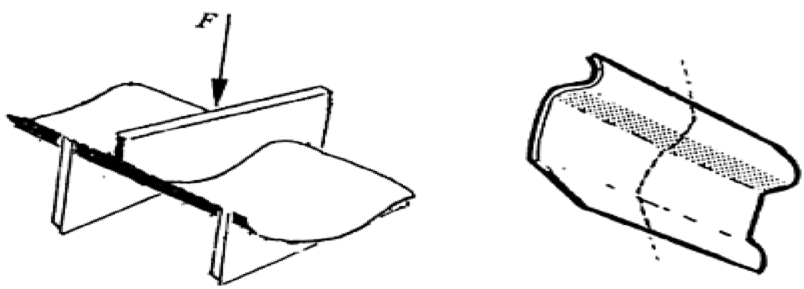

Figure 8. Test arrangement for the modified ITW/Parry method and the direction of crack propagation.

RESULTS AND DISCUSSION

A comparative result shown in Figure 9 shows that a much lower strength was obtained from the BS method in comparison with the other two arrangements. This is due to the nature of the loading and the shape of the tile. In the BS method the platen rests on the tile at two points where the loads are applied as point loads. The tiles failed in direct tension or through the development of local stresses at corners rather than in bending.

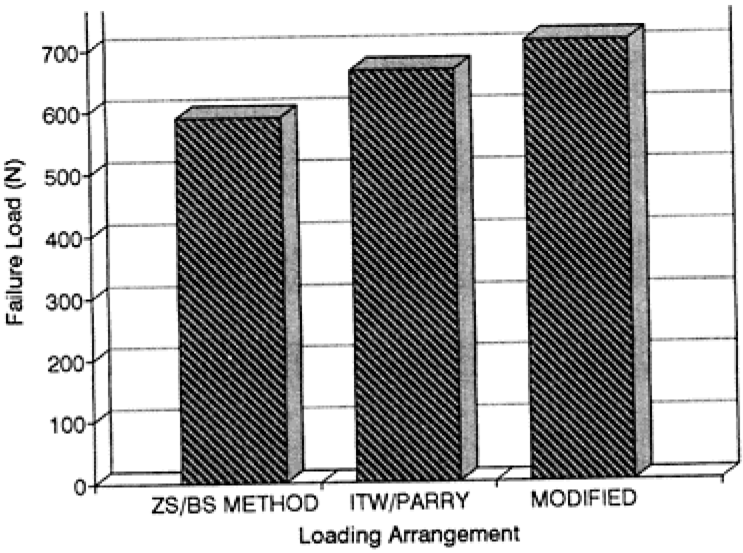

Figure 8. Average strengths from the three testing arrangements.

The BS method is designed to suit press extruded tiles. These tiles are comparatively flat in cross section and have the same overall height at several points as shown in Figure 9.

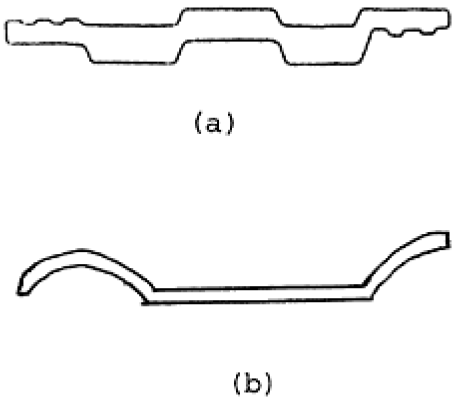

Figure 9. Cross-section of a) Typical press extruded tile , b) Roman II tile

The rest of the results were obtained from tests to the BS method. Figure 10 shows average test results at 28 days and 3 months for the tiles.

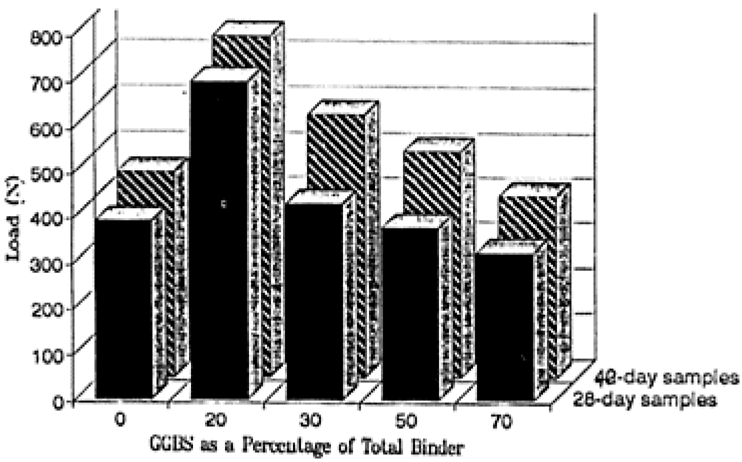

Figure 10. Failure loads at 28 days and 3 months.

It is generally observed that the best strength results were obtained for GGBS content of 20% for samples cured in the tank under the high temperature and humidity conditions. For samples cured at room temperature the strength generally decreased with increase in GGBS.

The BS transverse strength requires a minimum breaking load equal to 3.2 times the effective width (200mm for the Roman II tiles). This requirement is easily met in samples containing up to 30% GGBS at 28 days. The other samples could generally fulfil the BS requirement at older ages.

Another difference between the test methods is the fracture mechanism. In the BS testing the tile cracks and fails along the length of the tile while the ITW/Parry method fails across the width as shown in the sketch in Figures 6 and 7. The two methods, therefore, give cracks that are at right angles to each other.

When the tiles were being demoulded at 24 hours, some breakages were observed particularly for tiles containing high percentages of GGBS. It became obvious that more curing is required for such tiles. Samples with GGBS content of 30% or less did not experience handling breakages.

The permeability test showed a general tendency of increase in permeability with GGBS content. The use of admixture did not improve the permeability of the tiles.

The investigations have shown that the strength of the roofing tiles can be improved by replacing a limited amount of the cement with GGBS if the curing technique described above is used. It has also been demonstrated that the testing arrangement described in the BS 473 (2) does not give a true bending strength of the tiles. To obtain the strength in bending of the Roman II tiles, it is necessary to apply the load through a platen that matches the surface profile.

The use of water reducing admixtures is not beneficial to the tiles as regarding the durability and water permeability.

More work is currently being done to smooth out the strength curve between 0% and 30% GGBS content and also to investigate the influence of the fibre reinforcement on the increase in strength.

ACKNOWLEDGEMENTS

The author wishes to thank Parry Instamac (Pvt.) Ltd. Zimbabwe and the director John Duxbury for making the tiles and for information received.

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MICROPORE STRUCTURE VARIANCES DURING HYDRATION OF CEMENT PASTES

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ABSTRACT. Pore size distribution of several types of hydrated cement pastes has been studied by means of mercury porosimeter. The tested types of cements were the following: Ordinary Portland cement, Blended cement with 30% Sandorin Earth (pozzolan) Blended cement with 30% Ptolernais fly ash (CaO rich) and Blended cement with 30% Megalopolis fly ash (CaO and SiO_2 rich). The samples were cured in several baths as water bath, sea water bath, Mg^{2+}-bath and Cl—bath. The evolution of pore size distribution in the mentioned cement pastes, was related to the time of hydration. It is concluded that the pore volume of the micropores that have radii among 45 and 2500 Å is a function of the time of hydration. Relationships based on statistical analysis of micropore structure have been proposed to ensure the relation between the pore volume and the time of hydration of the above cement pastes.

Keywords: Ordinary Portland cement (OPC), Blended cements, Pozzolans, Fly ashes, Hydration of cement, Aggressive media. Pore sizes Distribution.

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INTRODUCTION

Both practice and research have already demonstrated that porosity and pore size distribution mainly affect some physical and mechanical properties of hardened cement pastes. Various formulas have been proposed concerning relationships between compressive strength and porosity [1]. Several studies are also referred to the durability and permeability of concrete related to its pore structure [2]. Curing conditions as well as composition of the pastes in particular mineral admixtures influence total porosity and pore size distribution [3].

It is also reported that cement pastes containing pozzolans have greater porosity than those made from Ordinary Portland Cement [4]. B.K.Marsh et al [5] suggested that making cement mortars with seawater instead of fresh water intails a greater pore volume in pore diameters less than 100 A and lower volume in the diameter range of 100–10000 Å.

The purpose of this study has been to study the pore size distribution of several cement pastes during their curing with different baths such as fresh water, sea water and solutions containing Cl^{-} and Mg^{2+}.

EXPERIMENTAL

Materials, Mixing, Curing and Sampling

Four cements were used consisted of Ordinary Portland Cement (O.P.C) and its blends with 30% of each of the following Pozzolans: Sandorin Earth (S.E) Megalopolis Fly Ash (M.F.A) and Ptolemais Fly Ash (P.F.A) and named C1, C2, C3 and C4 respectively. The chemical composition of the materials used, as well as their fineness are shown in Table 1.

Table 1. Analysis of the used Raw Materials

Oxide, %	O.P.C	S.E	M.F.A	P.F.A
SiO_2	19.65	55.90	45.52	37.02
Al_2O_3	4.73	17.28	16.69	16.80
Fe_2O_3	3.85	9.60	9.58	6.61
CaO	63.91	7.84	16.52	27.21
MgO	2.32	3.70	2.50	3.24

SO_3	2.84	traces	4.66	5.41
K_2O	0.51	1.22	1.78	1.25
Na_2O	0.25	3.00	0.59	0.49
L.O.I.	1.25	1.30	1.21	2.74
Fineness (cm^2/g)	4100	6850	6650	5500

Pastes were prepared from these cements in an appropriate mixer, using a W/C ratio of 0.25, 0.27, 0.30 and 0.33 for the cements C1, C2, C3 and C4 respectively, as determined by standard consistency test (ASTM C-187). The pastes were moulded in bars 160mm x20mm x20mm and cured for 24 hours at 20±2°C and R.H=90%. After that the pastes were cured for 7, 28, 90, 180 and 360 days in the following curing baths, at 20±2°C:

a. Fresh water.
b. Aq Solution of 2000 ppm Cl^- ions, coming from dissolution of NaCl.
c. Aq Solution of 2000 ppm Mg^{2+} ions, coming from dissolution of $Mg(NO_3)_2$.
d. Aq Solution of 2000 ppm Cl^- ions, 3000ppm SO_4^{2-} ions and 2000 ppm Mg^{2+} ions, coming from dissolution of NaCl, Na_2SO_4 and $Mg(NO_3)_2$ respectively and named "Seawater".

At predetermined ages samples of the prepared pastes were taken from each bath in order to examine their porosity. The hydration was interrupted by crushing and washing them with acetone, diethylether and vacuum drying at room temperature for 24 hours in a free CO_2 atmosphere.

Pore Size Measurements

The pore size distribution analyses were carried out by means of a mercury porosimeter technique. The test was concerned with the distribution of the pore volume in correlation to their radii. The porosimeter was capable of determining pore size distributions down to 45 Å.

Assuming that all the pores are cylindrical, pore radii were determined on the basis of the well-known Washburn equation:

$$r=-2.\sigma.\cos\theta/p$$

Where:

r=pore radius (E); σ=surface tension of mercury (=484 dynes/cm); θ=mercury contact angle (=141°) and p=absolute pressure exerted (kg/cm^2).

Our interest is focused to the pores of radii up to 2500 Å. It was found that, for pore radii greater than 2500 Å the porosity measurements were remarkably affected on the particles size of the sample.

RESULTS AND DISCUSSION

Figure 1 and Figure 2 present the average pore volume distribution for the cements and curing baths used at the predetermined ages of hydration. In these figures four characteristic intervals are observed, referred to the ranges 45–100 Å, 100–550 Å, 550–1100 Å and 1100–2500 Å, where the variance dVp/Vp is remarkably varied.

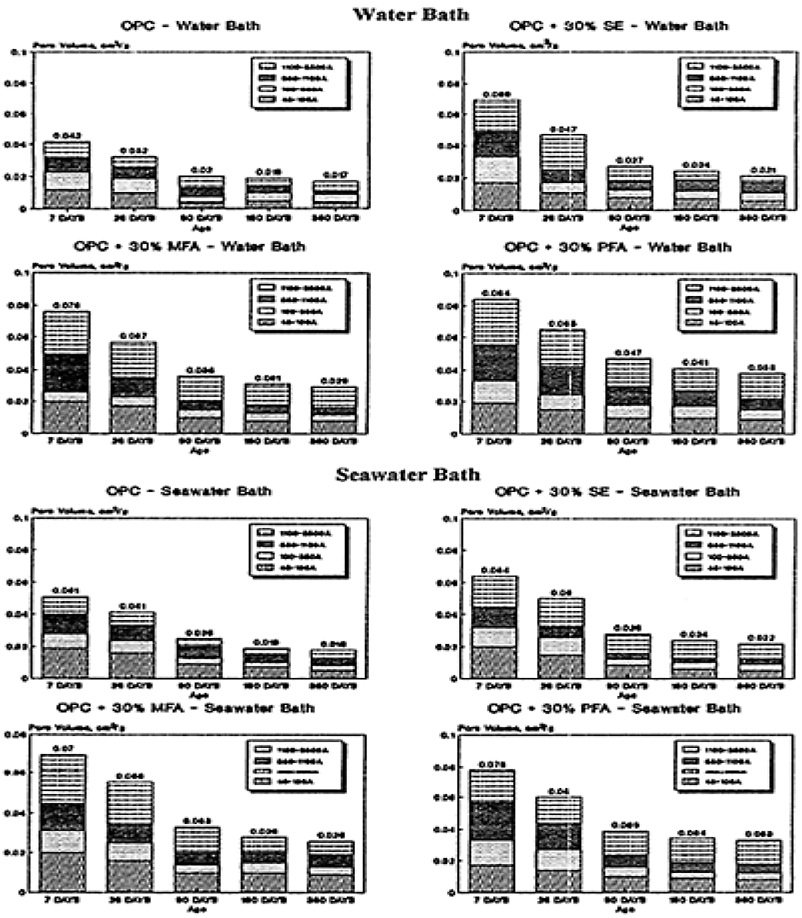

Figure 1. Average Pore Volume Distribution.

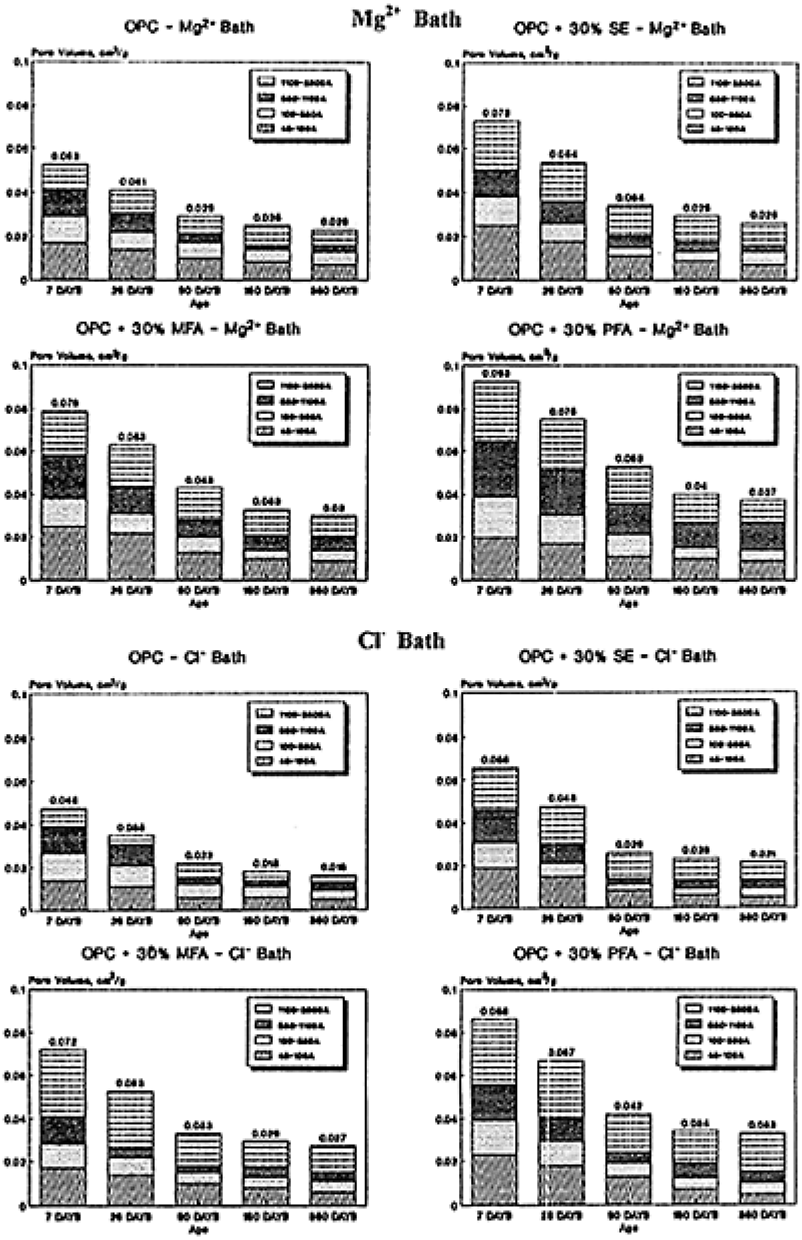

Figure 2. Average Pore Volume Distribution.

It is clearly observed also that the time of hydration influences significantly the structure of porosity in all the cement pastes and curing baths used in the studied range of pore radii (45–2500 Å).

From these figures, it is observed that the time of hydration of the cement pastes affects on the structure of porosity in the range of pore radii 45 Å and 2500 Å. The relationship between time of hydration and pore volume is shown in Fig. 3, Fig. 4, Fig. 5 and Fig. 6. The strength of the relationship found between time of hydration and pore volume has been evaluated statistically by means of regression analysis. These relationships can be expressed by the following equations:

a. Water Bath

$Vp^e=k/(t+m)$ (Figure 3—Curves a and b)

the coefficients e, k and m range among the following values (curves a and b respectively):

$e=3.93–4.65$, $k=3.18\times10^{-5}–7.82\text{x}10^{-5}$ and $m=1.16–0.96$

b. Sea-water Bath

$Vp^e=k/(t+m)$ (Figure 4—Curves a and b)

the coefficients e, k and m range among the following values (curves a and b respectively):

$e=3.38–4.07$, $k=37.17\times10^{-5}–22.96\text{x}10^{-5}$ and $m=0.22–0.39$

c. Mg Bath

$Vp^e=k/(t+m)$ (Figure 5—Curves a and b)

the coefficients e, k and m range among the following values (curves a and b respectively):

$e=4.49–3.90$, $k=1.29\times10^{-5}–82.9\times10^{-5}$ and $m=1.06–0.22$

d.Cl Bath

$Vp^e=k/(t+m)$ (Figure 6—Curves a and b)

the coefficients e, k and m range among the following values (curves a and b respectively):

$e=3.19–3.77$, $k=55.4\times10^{-5}–74.1\times10^{-5}$ and $m=1.30–0.05$

Where,

t is the time of hydration, [days]; **Vp** is the pore volume, [cm^3/g]

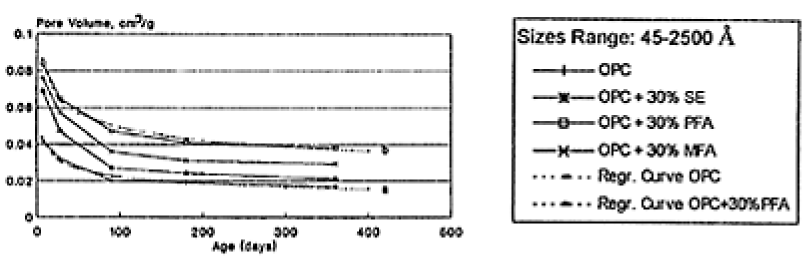

Figure 3. Water bath

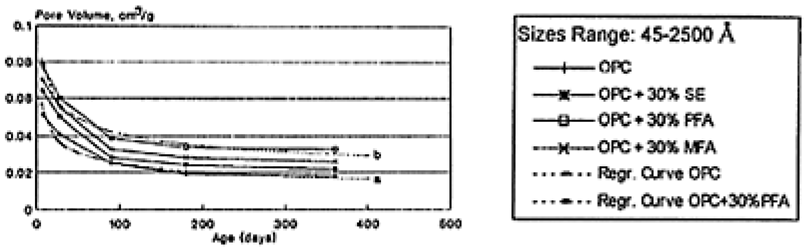

Figure 4. Seawater bath

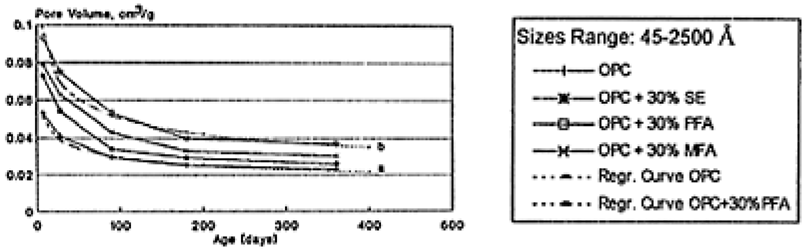

Figure 5. Mg^{2+} bath

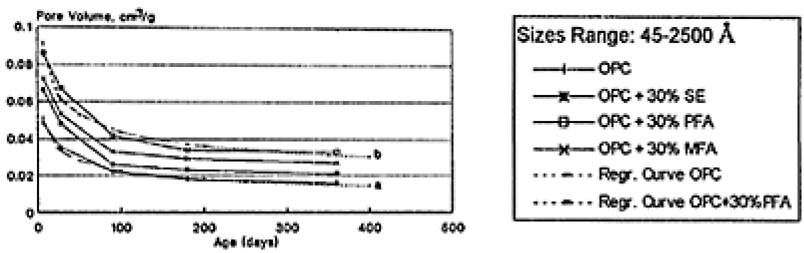

Figure 6. Cl^{-} bath

Figures 3, 4, 5 & 6. Pore Volume—Time of Hydration

CONCLUSIONS

Evaluation of results presented in this paper leads to the conclusion that pore structure, especially referring to the range of pore radii between 45 and 2500 A, is a function of the hydration time. Furthermore, relationships based on statistical analysis of the test results, are given for the estimation of this relation between time of hydration and pore volume distribution.

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IMPROVING THE PROPERTIES OF ADOBE BY POZZOLANIC MATERIALS

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Appropriate Concrete Technology. Edited by R K Dhir and M J McCarthy. Published in 1996 by E & FN Spon, 2–6 Boundary Row, London SE1 8HN, UK. ISBN 0 419 21470 4.

ABSTRACT. Urbanization is a common problem especially in developing countries, as more and more poor people migrate to the cities. The basic building fabric in most of the dwellings and houses constructed by these people is earth. Earth is one of the oldest building materials known to man. Many techniques used today have changed little from early methods. Most of these methods make use of local resources, are labor intensive, and are logical and effective. However, it is a common knowledge that adobe buildings are not sufficiently resistant to the destructive action of nature, especially water. A research study was performed to evaluate pozzolanic mixtures that incorporate brick powder, lime and fly ash combinations for improving the mechanical and physical properties of adobe bricks and preservation of adobe walls. The test results showed that most of the mixtures are economical with adequate strength and durability.

Keywords: Adobe, Fly ash, Brick powder, Pozzolanic reaction

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INTRODUCTION

The technology of building with earth was developed by trial and error from the earliest beginnings of humankind. Specific methods vary with geographic areas and cultural tradition, but are basically the same. Most of these methods make use of local soil, are labor intensive, and are logical and effective [2].

Unfortunately, adobe structures are severely affected by nature due to deficiency of earthen fabric. There are many individual factors that contribute to the decay of earthen architecture, but by far the most destructive is slow decomposition over time due to weathering.

The use of natural unstabilized adobe plasters to preserve earthen architecture against destructive action of water has produced unsuccessful results. Also the traditional mortars incorporating binders such as cement, lime and gypsum have different physical and mechanical properties than earth and are not harmonius with the original structure. Also many of the chemical amendment methods had limited successes with various disadvantages [3].

The objective of this research was to develop durable compositions incorporating pozzolanic materials for the protection of earthen material against rain. Recent research has been inspired by a historical binder used before invention of cement, throughout the centuries in Anatolia by different civilizations. This water resistant binder was named “Horasan” and composed of volcanic or wood ash, fired earth powder, lime and water. In some references egg-white is also included in this composition. However, there is no evidence concerning the exact ratios of the ingredients.

Materials

In this study, pozzolanic materials such as fly ash and brick powder were incorporated to the local earth to improve the durability of plain soil. Hydrated lime was also added to the mixtures to activate pozzolanic reactions.

The pieces of irregular or damaged fired bricks were collected, crushed and reduced to fine powder size. Hydrated lime procured in commercial paper sacks.

The other least known ingredient, fly ash, is a by-product of the coal combustion process at power plants. To prevent air pollution caused by thermal plants, electrostatically precipitated fly ash accumulates daily throughout the world in enormous quantities. It is the first rating waste in size, for most of the industrial countries. This quantity of waste causes serious environmental, technical and economic problems that need to be solved. Fortunately, fly ash can be effectively utilized since it will combine with lime to produce a cementitous material in presence of water [1].

In the present study, fly ash samples from Soma B thermal plant were used in the experiments. This plant consumes 1.75 million tons of low calorie lignite (2400 kcal/g) every year, with a 41 % C-type ash output. The chemical and physical properties of the ash are shown in Table 1.

Local soil is used in the fabrication of adobe bricks, adobe walls and in the plaster compositions.

EXPERIMENTAL DETAILS

Various amounts of fly ash and lime were added to brick powder start pozzolanic reactions and to increase the mechanical, physical and workability proporties of the compositions.

Table 1. Chemical Analysis & Physical properties of Soma B fly ash

Compound	Chemical Comp. by wt. (%)	Property	Value
SiO_2	47.40	Specific gravity	2.39
Al_2O_3	25.10	Min. unit wt.	1.00 g/cc
Fe_2O_3	6.80	Comp. unit wt.	1.18 g/cc
CaO	12.10	% passing # 40 sieve	98
MgO	1.44	% passing # 100 sieve	96
SO_3	1.60	% passing # 200 sieve	92
TiO_2	0.60	% passing # 325 sieve	84
Na_2O	0.10	Specific surface	2830 cm^2/g
K_2O	0.50	Pozzolanic activity	
P_2O_5	0.30	Index	8.2 N/mm^2
loss on ign.	1.1		
Undet.	2.96		

Dry and wet mixing procedures of the mortars were accomplished by means of a homogenizer and a Hobart mixer to ensure uniformity. Fabrication of the specimens was accomplished by table vibration in steel forms. The specimens were then stored under laboratory conditions until testing. Standard 50 mm cube specimens were tested to determine their 7th and 28th days compressive strength values. The tests were repeated on duplicate specimens soaked in water for four hours.

Plaster mortars and their respective mechanical properties are shown in Table 2. As can be observed from Table 2, the mechanical properties of the duplicate specimens did not decrease meaningfully ever after four hours of soaking in water. The compressive strength values are more than required values for the job. (The generally required compressive strength for adobe bricks is about 1 N/mm^2.)

The second phase of the research project, was developing pozzolanic plasters for adobe walls.

Table 2. Compressive Strengths of Various Compositions

Mix No	Soil (%)	Lime (%)	Fly (%)	Ash DRY COMP. STR. (N/mm^2)		WET COMP. STR. (N/mm^2)	
				7 DAYS	28 DAYS	7 DAYS	28 DAYS
1	–	3	12	4.2	8.3	3.9	8.2
2	–	6	9	3.7	7.7	3.5	7.5
3	–	7.5	7.5	3.6	7.2	3.4	7.3
4	–	9	6	2.9	5.8	2.5	5.4

5	–	12	3	2.8	4.7	2.5	4.3
6	10	3	12	3.1	4.4	3.1	4.0
7	15	3	12	2.1	3.4	1.8	3.3
8	20	3	12	1.5	2.8	1.4	2.2

A composition of 3% lime, 12 % fly ash and 85 % brick powder was taken as a base mixture for plastering amendments. By adding various percentages of soil to this reference mixture, mortar specimens were prepared and tested with the same procedure described above.

Plaster mortars and their respective mechanical properties are also shown in Table 2.

The compositions mentioned above were implemented in the plastering of small scale walls were subjected to simulated rain to study their resistance to erosion.

The test walls were subjected to 2 hours of continous simulated rain during five day time periods. An adobe wall specimen plastered with plain soil was also tested for comparison. Two specimens were tested for every pozzolanic mixture.

After completion of simulated rain cycles, damage suffered by the specimens was visually rated as light, moderate or severe. The test results are shown in Table 3.

As can be observed from Table 3, pozzolanic plasters were much durable than plain soil plasters. For example, Mixture No. 1 visually showed almost no damage during exposure to rain simulation of 10 cycles, where as plain soil plaster almost disintegrated after 5 cycles.

Table 3. Results of Simulated Rain on Test Walls

Plaster No.	Number of Cycles	Damage
P (Plain Soil)	5	Severe
1	10	No damage
6	10	Light
7	10	Moderate
8	10	Moderate

CONCLUSIONS AND RECOMMENDATIONS

From the preceeding test and information gathered, the following general observations and conclusions can be stated:

a The generally required compressive strength for adobe bricks is about 1 N/mm^2. Even the lowest compressive strength of the test series is higher (1.4 N/mm^2) than this value.

b The main hazardous effect on adobe bricks or earthen architecture is water. This deficiency may be overcomed by the use of pozzolanic mixtures in brick manufacturing or in plastering jobs. The tests performed on small scale specimens and rain simulation tests proved the durability of the compositions against water effect.

c The color of the mixtures can be arranged by incorporating local soil to the mixtures. It should not be forgotten that, the high percentages of soil have negative effects on durability and mechanical properties of plasters.

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Theme 5
NON-FERRROUS REINFORCEMENT

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Leader Paper

Tailoring the Properties of Concrete Structures with Appropriate Non-Ferrous Reinforcements

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TAILORING THE PROPERTIES OF CONCRETE STRUCTURES WITH APPROPRIATE NON-FERROUS REINFORCEMENTS

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Appropriate Concrete Technology. Edited by R K Dhir and M J McCarthy. Published in 1996 by E & FN Spon, 2–6 Boundary Row, London SE1 8HN, UK. ISBN 0 419 21470 4.

ABSTRACT. There exist a high selection of non-ferrous fibres, which have been applied into fibre reinforced concretes (FRC). The fibres are either natural plant fibres or synthetic fibres. However the use in practice still is quite limited. The aim of the future development work will be to avoid the difficulties, which still exist regarding to the long term durability and some times also regarding to the mechanical properties of FRC. It is proposed to apply a systematic tayloring of FRC in order to meet the requirements, which arise in the use at each application area. The local requirements of users and the requirements of the whole environment are quite different in different parts of the world. Also the buildings own different types of parts: Both strong and long life parts like the bearing frames and soft shorter term parts like the infill. The non-ferrous FRC materials are specially suited for the soft parts, but can be applied as a part of hybride reinforcements also into the strong parts. The principle of tayloring of the materials and structures leads to a design process, where specially the durability design, environmental analysis and generally the service life assessment have an important role.

Keywords: Fibre reinforced concrete, non-ferrous fibres, durability, design methods, tayloring of materials, service life design.

Professor Asko Sarja is a research professor in structural engineering and system building at Technical Research Centre of Finland (VTT). Since 1970 he has worked in several fields of structural engineering maynly in the research but beside the main job also in education at the Technical University of Helsinki. The working fields are mechanics of structures, reliability theory of structures, concrete technology and industrialised building technology including CAD. He has written more than 200 articles and publications, many of them in international magazines and books. He is actively working in international organisations like RILEM, CIB, IABSE and FIP.

INTRODUCTION

Since 1970's s extensive research has been done on fibre reinforced concretes (FRC). A lot of promising innovations have been achieved in laboratories and in the first applications. However, the practical applications have been quite limited. The practical use has remained so limited partly because of the uncompetitive price of the FRC materials and structures, and partly because of the unsatisfactory properties of FRC, e.g. regarding its durability. However, we can be satisfied that there is great potential for successful applications of FRC in many different areas. Besides the further development of the material, a casewise systematic performance-based analysis of the requirements and the optimised structural innovation interactively with the specification of material parameters and with the choice of the FRC material can be the key to wider and more useful practical applications of FRC.

Designing the structures and their materials according to the performance requirements of different applications can be called the tayloring of the material. Concrete as such already possesses great versatility and potential for modification through the mix design and through the structural design and detailing. When adding fibre reinforcement, the alternative modification possibilities become vast. The challenge for both the materials engineers and for the structural engineers is the optimal utilisation of these vast alternatives. In research, the different requirements must be clearly realised and classified. After that the researches must produce for the designers calculation models in order to enable the controlled and optimal tayloring of the structural properties during the design stage.

The basic requirements of the material properties in structures vary greatly. In large civil engineering structures like bridges and dams strength and durability are of the highest priority. Buildings typically have quite different requirements in different parts of the building. Strength, fire safety and durability are the dominating requirements in the building frames in order to guarantee the long service life and safety of the building. Visual appearance, thermal and moisture performance, durability, toughness and moderate strength are the main categories of requirements for facades. The infill structures must possess quite different properties; they could be called the soft parts of the building. Controlled sound insulation, healthiness, fire safety, moderate strength and easy fastening of furniture and other equipment are typical performance criteria for the infill structures.

For economical production and for flexibility in alterations during use, several parts of the building like the infill must have good workability properties during and after construction. The good workability includes the workability of the concrete mass during mixing, transport, handling, casting and compacting, but also easy workability of the hardened structures through sawing, drilling, nailing, screwing and bolting. The workability properties are especially important for the materials and structures which are aimed at renovation purposes.

In order to be able to meet the challenge of a controlled and optimised structural design for performance requirements, the designer must have the methodological capability and a deep knowledge of the properties of different FRC materials. Also the researchers can benefit from the performance- based systematics in innovative and effective tailoring of the materials and in the innovative developments of the structures and their design methods. Besides the traditional mechanical, physical and chemical aspects, the health-related properties and environmental impacts must also be taken into account.

OVERVIEW ON THE TYPES OF FIBRE REINFORCED CONCRETES

Fibre reinforcements

Fibre reinforcement can be introduced into concrete in different forms, as shown in Figure 1 /1/. Thus, the FRC composite material can vary owing the reinforcement from totally dispersed fibres to very sophisticated layered, directed or shaped fibres. Additionally, different bundled and woven fibres are used.

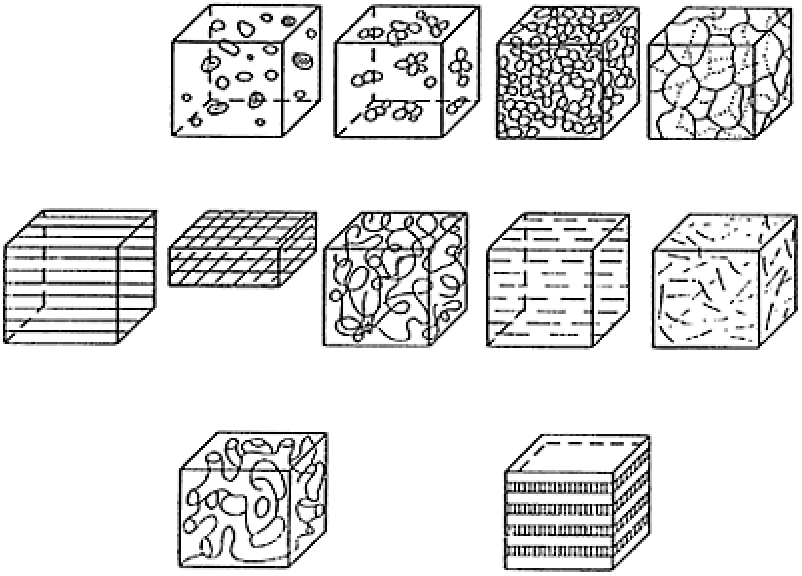

Figure 1. Examples showing the structure of different fibre reinforced concrete materials.

Applying the fibre reinforced concrete, different structural compositions can again be manufactured, as shown in Figure 2 /1/.

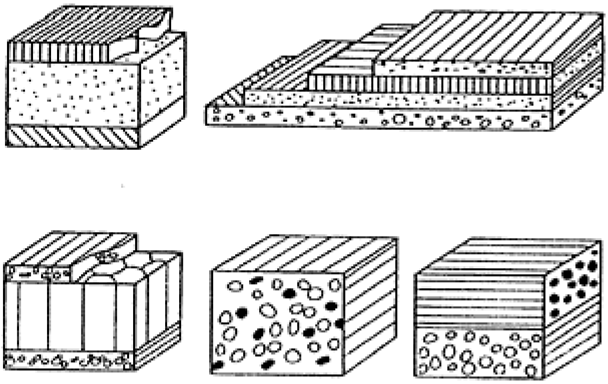

Figure 2. Examples of different types of fibre reinforced composite structures.

Classsification and properties of the fibres

The most common non-ferrous fibres can be classified into synthetic and man-made fibres. A flow chart of organic fibres is shown in Figure 3 /2/.

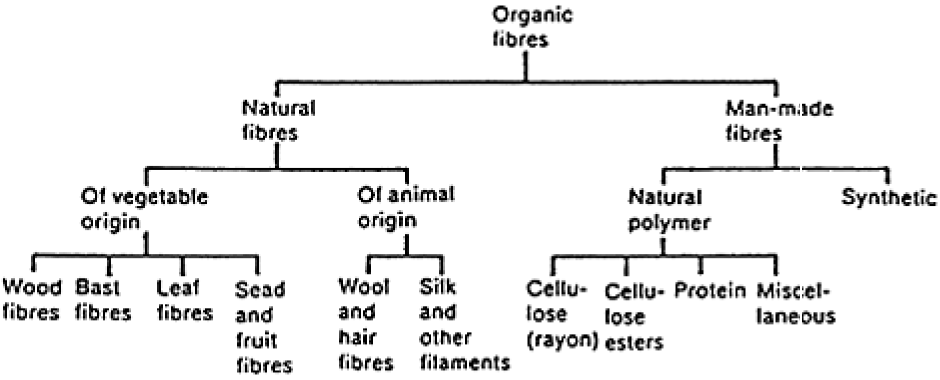

Figure 3. Fibre classification chart.

Synthetic fibres mean all types of fibres which are produced in industrial chemical processes. Most often these materials are different types of polymers, carbon or glass. Amorfous steel is also presented here, although it belongs to ferrous materials. A list of common commercially available synthetic fibres and their properties are presented in Table 1 /3/. The properties presented in Table 1 are only estimates, because the properties may vary somewhat depending on the manufacturer.

Table 1. Properties of some commercially available synthetic fibres.

Material Trademark (Manufacturer)	Diameter or thickness mm d_f	Length mm l_f	Tensile strength MPa f_{fu}	Youngs modulus MPa E_f	Max. elongat. %	Density kg/m^3	Max. temp.C
AR-glass Cem-Fil (Cem-Fil ltd.)	0.02	max. 50	1 700	72 000	2.4	2 680	538
Carbon Kreca C-103T (Kureha Chem. ind) Tenax HTA-C3E (Akzo)	0.018 0.007	3 3	590 3 400	30 000 238 000	2 1.4	1 650 1 780	
Aramid Twaron T 1095 (Enka/Akzo)	0.012–0.018	0.4–3.0	2 800	80 000	3.3	1 440	500
Polyacrylonitrile Dolanit T11 (Hoechst AG)	0.052–0.104	6–24	410–650	15 340–18 900	6–9	1 180	150
Polyvinyl alcohol Kuralon RF350 Kuralon RF 1500 Kuralon RF4000 (Kuraray Co ltd)	0.2 0.4 0.66	12 30 30	900	29 000		1 300	180
Polypropylene Krenit Special (Danaklon A/S)	0.035x 0.25–0.66	6,12	340–500	8500–12500	8–10	1 010	145
Amorfous steel Fibraflex (Pont-a-Mousson)	0.03× 1.0–2.0	15–60	2 000	140 000		7 200	

Some properties of natural fibres are presented for wood fibres in Table 2 /4/ and for the fibres from other plants in Table 3 /2/.

Table 2. Properties of wood fibres.

Fibre	Density (g/cm^3)	Diameter (μm)	Length (mm)	Tensile strength (MPa)	Modulus of elasticity (GPa)
Cellulose					
Wood, straw (dry)	1.5	15–40	0.5–4	300–500	8–10

Sisal	1.5	10–50	1000	800	30
Cotton	1.5	10–30	60	300–900	10
Kraft cellulose, unbleached *(Pinus)*	1.5	15–60	1.2–2.7	200–1500	40
Kraft cellulose, bleached *(Pinus)*	1.5	15–60	1–2.7	200–1300	40
Thermo-mechanical pulp	1–1.5	20–70	1.5–2.5	200–800	
Kraft cellulose, bleached (birch)	1.5	18–60	1.0–1.2	200–1000	5–40
Kraft cellulose, bleached (eucalyptus)	1.5	12–30	0.9–1.2	200–1 300	45

Table 3. Properties of some natural plant fibres.

Fibre	Colour	Texture	Specific gravity	Water absorption (24h), %	Tensile strength (kg/mm^2)	Modulus of elasticity (GN/m^2)	Elongation at break (%)	Electrical resistivity ($\times 10^5$ Ω cm)
Coir	Golden brown	Smooth and tough cylindrical and twisted fibre	1.31	80	9–14	4–6	15–40	9–14
Sistal	Dirty white	Long, rough and twisted fibre	1.35	150	100–200	34–62	3–7	4.7–5.0
Jute	Light yellow	Soft and resilient fibre	1.36	250	400–500	17.4	1.1	–
Bamboo	White to light yellow	Smooth to rough fibre	1.50	–	–	–	–	–
Banana	Light yellow to light brown	Smooth to rough, long cylindrical fibre	1.30	400	110–130	200–510	1.8–3.5	6.5–7.0

Aak	Dirty white	Soft, cotton-like	1.17	350	74–87	–	–	–
Bhabar	Dirty green	Rough and brittle fibre	1.30	185	5–70	–	–	–
Castor	Slaty white	Short crumbled soft fibre	1.01	235	25–40	–	–	–
Dhakala	Yellowish white	Bundle of soft fibres	1.06	120	40–85	–	–	–
Hemp	Light yellow	Soft and resilient fibre	1.36	140	40–200	–	–	–
Munja	Creamy yellow	Hard and brittle fibre	1.29	160	20–75	–	–	–
Shinio	Greyish white	Short, rough crumbled and twisted fibre	0.94	230	30–55	–	–	–
Khimp	Dirty yellow	Short bundle like cotton lint	1.33	450	60–65	–	–	–
Pineapple leaf	Creamy white	Soft with pointed ends	1.44	–	360–740	24.3–35.1	2.0–2.8	–
Palmyrah	Yellowish	Stiff, thick, strong	–	–	180–215	4.4–6.1	2.0–2.8	–

PRINCIPLES OF THE DESIGN OF FRC STRUCTURES

Calculation models for the design of FRC materials and structures

For the tayloring of the materials and structures for specified performance properties, calculation models of the relevant properties in each case must be used. For FRC there exist both general models, which are valid for different kinds of concrete composites, and specific models, which are valid for some special composites. Most of the calculation models deal with the mechanical properties, but some models exist also for physical properties.

The leading principle for the modelling of FRC is the general law of mixtures /3/

$$P_c^k = \sum_i \eta_i V_i P_i^k \tag{1}$$

where P_c is the property of the composite

P_i the property of the ingredient

k the combination factor, −1<=k<=1 (+l=parallel, −1= serial)

η_i the efficiency factor for P_i

The volymetric value V_i can be calculated from the mass values with the equation

$$V_i = \frac{M_i}{\gamma_i} \tag{2}$$

where M_i is the mass of the material and

γ_i the density of the material

Applying the parallel model of the law of mixture, e. g. the modulus of elasticity of the uncracked composite and the cracking tensile strength can be calculated from the equation

$$P_c^k = \sum_i \eta_i V_i P_i^k \tag{3}$$

$$f_{fccr} = f_{ctu}(1-V_f) + \eta_\Phi \eta_l \sigma`_f V_f \tag{4}$$

where E_{fc} is the modulus of elasticity of the composite material

E_c modulus of elasticity of the concrete matrix

E_f modulus of elasticity of the fibre

V_f volume fraction of the fibre

η_Φ orientation efficiency factor

η_1 fibre length efficiency factor

$\sigma`_f$ stress of the fibre=$E_f\varepsilon_f$ <= $E\varepsilon_{ctu}$

ε_{ctu} ultimate tensile strain of concrete

f_{ccr} cracking tensile strength of the composite material

After cracking, the basic law of mixtures (1) is no longer valid, but the separation effect of the crack must be taken into account. For cracked FRC the most important aim is to limit the crack width and crack length with the fibres in order to avoid damage to the structure through the separation of the parts between the cracks. This leads to the critical amount of fibres V_{crit}, which can be calculated with the equation /5/

$$V_{crit} = \frac{E_{fc} f_{ctu}}{E_c \eta_\theta \eta_1 f_{fu}} \tag{5}$$

where f_{ctu} is the tensile strength of the concrete matrix and

f_{fu}, tensile strength of the fibre

The mechanical meaning of eq. 5 is that the fibres can take all the force which is moving from the matrix after the tensile rupture of the matrix. When designing the composite material using this equation it is important to use the upper limit value of f_{ctu} and the lower limit value of f_{fu}, because the design otherwise leads to an unsafe condition and to the progressive rupture of the material at the cracks.

The critical length of the fibre, when the slipping begins, is

$$l_{crit} = \frac{d_f f_{fu}}{2f_\tau}$$

where d_f is the diameter of the fibres or corresponding factor (=4•(fibre cross sectional area/ fibre cross sectional perimeter)

f_τ bond strength of the fibre in the matrix

After cracking, the tensile strength of the composite material is dictated by the fibres and can be calculated with the equation

$$f_{fctu} = \eta_\Phi \eta_l f_{fu} V_f \tag{6}$$

There exist different theoretical and experimental values for the efficiency factors of aligned fibres, 2-dimensional fibres and 3-dimensional fibres /3/,/5/,/6/.

Especially for individual types of composites, but also for some general uses, there exist also experimentally determined calculation models of the performance properties. These models often contain also physical, chemical and sometimes even biological properties /5/,/7/,/11/.

With the aid of the basic equations, different elastic and plastic calculation models can be used for the calculation of the bending and shear capacities of the FRC structures.

Experimental material characteristics and models

Theoretically produced material models mainly exist for the mechanical properties of the composites. For most of the physical, chemical and biological properties the experimental characteristics applying general standards are used /8/,/11/. Typical properties of this category are the weather resistance, frost resistance, permeability, fire resistance, insulation properties and appearance. The health-related properties and the environmental impact values are also of a similar character. Yhe interaction of several kinds of loadings is often of great practical importance. The high complexity of the composite material and of the deterioration phenomena often necessitates the use of several tests in order to guarantee that all factors are taken into account. This leads to the methodology recommented by RILEM /9/, as presented in Figure 6.

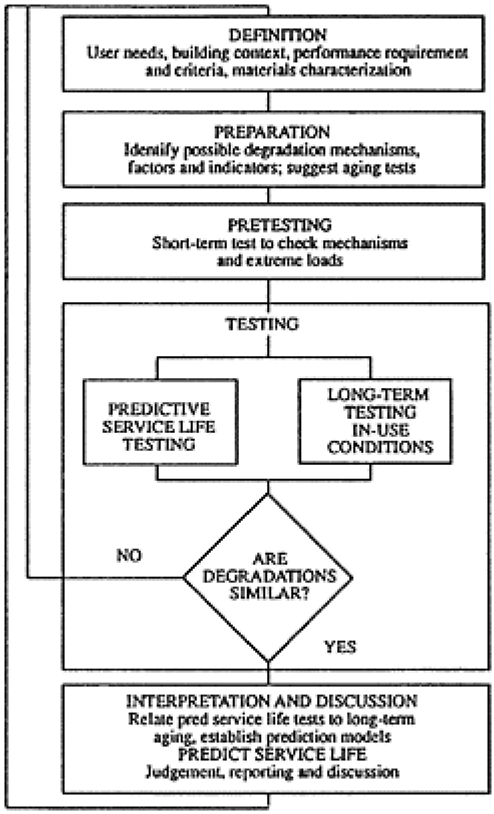

Figure 6. Methodology for science life prediction.

The characterisation of the FRC materials includes the properties of the ingredients and the properties of the interfaces /8/. The interface phenomena occur both at the interfaces between the cement paste and aggregate and between the concrete matrix and fibres. In the reserach and development of FRC materials the interface phenomena are the real key factors. A range of modern equipment and methods are now available for such interface research. There are many standardised testing methods for FRC materials. The application of multiple testing methods especially regarding durability properties is essential for reliable results /10/. Besides these special methods and calculation models also the methods and models of ordinary concrete can be used. A selection of calculation models on durability performance is presented in the design guide for concrete structures /11/.

A central characteristic factor is the toughness index, e. g. regarding the ASTM methods C 1018–92 and C 457–90 /12/. It is important to realise that the toughness often decreases remarkably with time because of the corrosion of fibres and because of the increase of tensile strength of matrix concrete and the strengthening of the bond between the concrete and fibres /12/.

INTEGRATED DESIGN METHODOLOGY

Integrating the application of the performance properties is made during the structural design stage.

The controlled service life and durability of the structures is of great importance when applying complicated FRC materials. For this purpose the corresponding methodology generally used for concrete structures can be applied /11/. The flow chart of the service life design is presented in Figure 7 /11/.

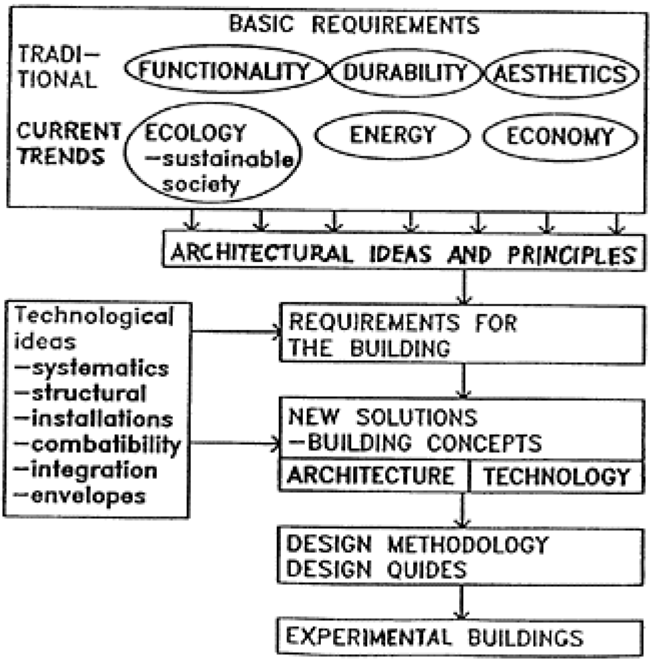

Figure 7. Flow chart of the service life design of concrete structures.

The integrated design includes multiple criteria decision-making, where the properties with different measures must be compared. This leads to quite wide discussions on the basic values when deciding the criteria for multiple optimisation.

Design procedure

The design of FRC structures follows the common design procedure, but some additional procedures are added in order to take account of the high number of alternative materials and their combinations in mix design. Special attention is payed also to the service life and to the cost effective optimisation of the material and structural properties for the entire service life of structures.

The design procedure includes the following phases, from which in each case the actual phases are chosen for the active use:

1. Identification of the mechanical, physical, chemical and biological loadings affecting the actual structures in their working environment during the service life.
2. Specification of the actual performance requirements and the limits and characteric values of environmental impact for the structures.
3. Selection of alternative constituents of the FRC, which possess the basic properties fulfilling the specified performance requirements.
4. Identification of the intended manufacturing processes (factory/site manufacture) and the detailed manufacturing methods.
5. Sketching alternative structural compositions and forms using the selected materials, with the objective of meeting the specified performance properties, environmental impact limits and manufacturing requirements.
6. Specification of alternative mix designs for the specified performance requirements utilising selected calculation models of material properties corresponding to the identified loadings.
7. Application of the mix designs the sketched structural compositions, thus designing alternative structural solutions.
8. Comparison of the alternative structures regarding the specified performance requirements and taking into account the environmental impact of the alternative structures during the entire service life.
9. Selection of the structural solution and the materials for the final design.
10. Final design and detailing of the structure.

Durability design

The design procedure at the durability design stage follows the following phases /11/:

1. Specification of the target service life and the design service life
2. Analysis of environmental effects the structure
3. Identification of durability factors and degradation mechanisms
4. Selection of a durability calculation model for each degradation mechanism
5. Calculation of durability parameters using available calculation models
6. Possible updating of the calculations of the ordinary mechanical design
7. Transfer of the durability parameters the final design

The entire procedure is quite complicated. For that reason it can be simplified in practical cases using the expertise and earlier experiences of the designer. The methodology can be followed at the product development stage of new prefabricated units or of new building types for specific uses.

PRACTICAL APPLICATIONS AND MANUFACTURING METHODS

Products and manufacturing methods

In the literature there are many examples of the practical use of FRC made of non-ferrous fibres /2/,/4/,/13/,/14/,/15/,/16/. However, there is the feeling that most of the reported applications are quite limited in use. Probably the most wide spread applications is the use in shotcrete material, where most steel fibres are replacing the earlier fabric reinforcement. Some real success stories of the FRC application are still awaited. Asbestos cement was such a candidate, but it failed because of health-related problems.

The application of new manufacturing methods is the real potential benefit of FRC. Several methods like extrusion, automated form casting and the Hatschek process are reported /7/,/13/,14/. The development of new manufacturing methods must be supported by materials and structural development of the products.

Design methods

At the mechanical design of non-ferrous reinforcements there are practical design rules and calculation methods for fibre reinforced concretes /5/,/6/,/7/,/17/,/18/,/19/, for non-metallic tendons /21/,/22/,/23/,/24/,/25/, for glued surface reinforcements /26/ and for fabric reinforcements /26/,/27/,/28/.

In durability design the general methodology and methods for the design of concrete structures can be applied /9/,/10/,/11/. The detailed degradation models needed for specific types of concrete structures with non-ferrous reinforcements are numerous /5/, /8/,/29/,/30/,/31/,/32/,/33/,/34/,/35/,/36/. This area is certainly the weakest with respect to existing quantitative calculation models. Therefore qualitative and approximative knowledge and testing of products continue to be used in most cases.

The analysis and optimisation of the environmental impact of structures can be calculated by applying the general methods and information available on the environmental profiles of submaterials /37/.

CONCLUDING REMARKS

Fibre reinforced concrete technology possesses an extremely high potential for advanced applications in both developed parts and developing parts of the world. In old industrialised countries the manufacturing can be automated in factories and mechanised on site works. In developing countries the manufacture often includes more manual works, which reduces the costs in those conditions.

There is a vast range of different fibre materials, either synthetic or natural materials. Local materials are often the cheepest and therefore preferred especially in the developing countries. The current trend towards ecological building also tends to favour the use of local natural materials even in developed counties. One promising possibility is also the application of hybride fibres and the combination of soft aggregate material together with fibres.

There has been and still are some problems regarding the long-term durability of non-ferrous FRC materials and products. The long-term durability of synthetic polymeric fibres is not always known for several decades, which is usually the service life requirement. The combined action of several severe loadings like temperature and moisture changes combined with weathering, frost attack, chemical weakening and mechanical loadings has caused the progressive growth of the cracks, leading to serious damages e.g. to facades. Traditionally, quite small fibre contents have been used in order to improve the economy of FRC. This is very dangerous in severe conditions, because the small fibre amount can not prevent the long-term progressive damage at the cracks. In order to avoid these kinds of difficulties it is important to apply efficient multiple testing procedures supported by theoretical calculations, comparisons and optimisations. Additionally, new materials and structural innovations and inventions are still needed in order to overcome the existing problems.

In the case of more massive structures like beams, columns and slabs the fibre reinforcement can be combined with bar reinforcement. In this case the role of the fibres is to take the secondary stresses, while the bar reinforcement takes the highest primary stresses and forces. The fibres can e.g. replace the stirrups, thus enabling the automated manufacture of the structures. Often the fibres can be concentrated in to the most stressed parts of the structures, thus helping to diminish the amount of fibres and the costs.

The nearly unlimited variations of FRC materials make it possible to tailor the materials and structural design for each purpose in order to meet the multiple requirements of the users and of ecological and economical aspects. This necessitates innovative product and production development as well as a design procedure in practical applications.

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DEVELOPMENT OF COVER METERS FOR STAINLESS REINFORCEMENT—TWO SUCCESSFUL APPROACHES

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ABSTRACT. Although the use of inherently non-corroding stainless reinforcement may reduce the criticality of the protective concrete cover, the need to control quality by measuring the location and cover to stainless bars is just as important as for conventional high-tensile steel. Unfortunately, cover meters based on magnetic methods will not respond to the non-magnetic steel; and even those meters based partly or wholly on conductivity response give signals which are much reduced and uninformative owing to the unusually low conductivity of stainless steel. It was therefore necessary to develop an instrument for the location and cover measurement of stainless reinforcement.

Two draft specifications were prepared, one with modest requirements, and the other considerably more exacting.

The first target was met by taking the technology that has been successfully used in the Protovale "Imp" stainless-steel wall-tie locator, and combining it with an enhanced version of the Protovale "Rebar Plus" bar locator.

The second target is being met by further development of the recently introduced "CoverMaster CM9" cover meter. The increased understanding of the dependence of the signal strength on bar diameter gained from the first model led to a new means of accurately determining the diameter of stainless steel bars, and in some instances the grade of steel also, and this ability is incorporated into the new instrument.

Both instruments, as a matter of practical necessity, also measure high-tensile rebars.

Keywords: Stainless steel, Reinforcement, Rebar, Cover, Covermeter, Non-destructive testing, Bar-sizing.

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INTRODUCTION

The Need for Cover Measurements

In conventional ferro-concrete a symbiotic relationship exists: the high-tensile reinforcement strengthens the concrete, and the alkaline concrete protects the steel against corrosion. However, situations are periodically met with where the desired quantity and position of reinforcement conflicts with the requirement for sufficient protective concrete cover. In these cases, the use of inherently non-corroding stainlesssteel reinforcement—whilst probably not the cheapest solution in the short term—is certainly the most logical solution (and sometimes the best long-term solution). At first glance, it might be supposed that the use of non-corroding reinforcement removes the need to check the final structure for minimum concrete cover; but further consideration suggests that the need to survey the reinforcement is in fact just as important, for a variety of different reasons:

Because the quantity of steel will almost certainly not have been over-specified, the presence of all bars should be verified;

For the same reason, the diameter and steel grade may need to be verified;

If the bars are not at their design nominal cover, they may not be in the correct positions to provide the intended tensile strength; and

Omission of reinforcement-checking implies a lower level of project quality control.

Unsuitability of Existing Cover Meters

Unfortunately, few—if any—of the many currently-available cover meters are suitable for stainless steel. All of the magnetic instruments based on the design of the original C&CA Covermeter [1], and also the newer “Streufeld” effect [2], give no signal at all from non-magnetic stainless steel. Even the more recent instruments- designed to respond either wholly or partially to the conductivity of the steel—yield a much-reduced signal strength, which makes location of small-diameter bars impossible and indications of cover meaningless; the conductivity of stainless steel is exceptionally low by comparison with other structural metals, and the inherently weak eddy-currents do not benefit from the “magnifying” effect of the permeability of magnetic steels. It was therefore felt necessary to develop a bar locator and cover meter for stainless steel.

Stainless-steel Cover Meter Specification

In fact, two target specifications were drawn up, one with modest requirements and the other with a much more exhaustive specification, and these are summarised in Table 1 which also briefly summarises the “solutions” developed.

Table 1 Target specifications for a stainless-steel cover meter

Requirement A	*Requirement B*
Modest maximum depth (say 70 mm)	Max. depth greater for larger bar sizes
Measure down to shallow covers (10 mm)	Measure shallower covers (<10 mm)
Work on standard bar sizes (8 to 40 mm)	Work on all bar sizes (6 to 50 mm)
Measure cover to bars of known size	Automatic bar-sizing (SS)
	Automatic grade determination
Accuracy better than ±3mm/8% on-site	Accuracy ±2mm/5% or better on-site
Display cover on single-scale analogue meter	Digital display of diam., grade, and cover
Audible output for bar location	Audible output for bar location
Also measure high-tensile bars	Recognise and measure high-tensile bars
	Automatic bar-sizing (HT)
Portable electronics, lightweight search probe	Portable electronics, lightweight search probe
Easy to use: only two control knobs.	Easy to use: menu-driven, on-screen prompts.
Solution A	*Solution B*
Combine the existing Rebar Plus bar locator with Imp Stainless Walltie detector; stabilise against battery voltage variations; add bar diameter compensation for both metals.	Enhance the existing microprocessorcontrolled CoverMaster CM9 cover meter; reconfigure sampling pulses to determine eddy current decay time and hence bar diameter and grade.
The “SuperStab”, which has been available for hire (and used successfully) since 1992 as the world’s first and only stainless-steel covermeter.	The “Asdic”; a laboratory version of which has been under test since 1991, and a site version of which is scheduled for introduction during 1996.

APPROACH “A” (MODEST)

Techniques taken from the Stainless-steel Wall-tie Locator

The metal-detection technique used is the Pulse Induction Eddy Current method. Rectangular pulses of current are passed through a search coil, with care taken to minimise the switch-off time. In the absence of any nearby conducting metal, the switch-off edge induces a large but brief transient voltage across the coil; after which, since the current and rate-of-change of current are both zero, the coil voltage is also zero.

If a conducting target is present within the field of influence of the search coil, the switch-off induces eddy currents to flow in the target, which initially are sufficient to regenerate the collapsed magnetic field. As there is no source of energy to maintain these eddy currents, they will decay away with a time-constant dependant upon the dimensions of the target (radius of the eddy current path) and the conductivity of the metal. These exponentially-decaying eddy currents generate a similarly-decaying secondary magnetic field which propagates back to the search coils and induces a small voltage which also decays exponentially. Example coil voltage waveforms are shown in Figure 1. The receiver circuitry of the detector must ignore the first few microseconds of coil voltage, which contain the large back-emf transient and also a rapidly-decaying signal component from ionic currents owing to any moisture in the medium in which the target is embedded.

For the detection of both mild and high-tensile steel, this "delay time" is typically in the region of 28μs; but the signals from stainless steel only persist for a time somewhat less than this (apart from very large diameter bars), so the instrument gives little or no response to small and medium sized stainless rebars. By reducing the delay time to between 16 and 18μs, an adequate signal can be received from most types of stainless steel wall-ties [3] and all sizes of stainless steel reinforcement. In practice, various other pulse widths and durations also need to be changed, so all pulses are derived from a single master clock which can be switched between two frequencies.

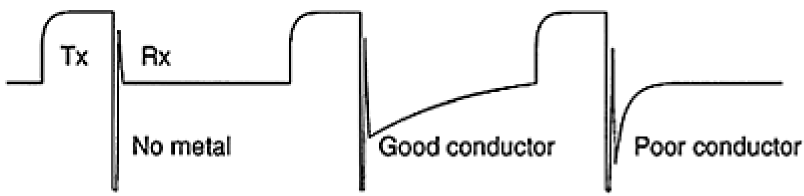

Figure 1 Coil pulse waveforms in the absence and presence of conducting metal

Enhancements to the "Rebar Plus" Bar Locator

The Rebar Plus is a bar locator which can also give an estimate of concrete cover [4]; it is not officially a cover meter (although its performance compares favourably with the older generation of "wooden box" cover meters). For simplicity, cover indication is by an analogue meter, and that scale is only calibrated for bars of medium size (12 to 20mm diameter); to achieve light weight, the unit is powered from just four small dry batteries, and the cover indication is somewhat dependent upon battery voltage.

For "solution A" the analogue meter was retained, but calibration for different bar sizes was introduced; and the new unit is powered by larger nickel-cadmium batteries, with all critical circuitry stabilised against battery voltage variation. A suitable method for compensating for differing bar diameters (as successfully used in the CoverMaster CM5 and CM52 cover meters [5]) is to control the gain of the receiver circuit, amplifying the signals from smaller bars and attenuating the signals from larger bars, so that the output signal to display is a function of cover but independent of bar size. This can be readily implemented by switching the value of just one feedback resistor in the receiver amplifier. However, to calibrate for eight different bar sizes (6, 8, 10, 12, 16, 20, 25 and 32 mm) for both high-tensile and stainless-steel bars, a total of sixteen such resistors are required. To remove the need for complex front-panel switches, and to avoid the need to route critical signals between the circuit-board and front panel, the resistor selection is actually performed by two 8-way solid-state multiplexers on the circuit board; a 3-bit bus selects bar diameter, and a fourth wire selects multiplexer and clock frequency and hence steel type.

A photograph of the completed instrument, provisionally named "SuperStab", is shown in Figure 2.

Figure 2 Photograph of "SuperStab": the first version of a stainless-steel cover meter

Site Applications of "Solution A"

The "SuperStab" was used by site inspection engineers from Site Services Testing (East Kilbride) to verify the positioning of the stainless-steel reinforcing bar within a large number of in-situ lintels in a Local Authority building at Livingstone, West Lothian, in early 1994; that building is now fully-commissioned and in service.

In the same year, it was used by John Mowlem Construction as part of their quality control of repair and refurbishment work on Grafton Court, a '60s concrete 15-storey tower block, in Hulme, Manchester, in order to establish the correct location of reinforcement to the balcony areas.

APPROACH "B" (THOROUGH)

Although the simpler model just described proved to be eminently suitable for locating and measuring stainless steel rebars, and is believed to be the first and only instrument in the world able to do so, it was decided to treat it as merely a test-bed for researching the necessary technology, and to develop a full-specification stainless-steel cover meter to achieve an even more exacting target. This new development can be thought of as an enhancement of the recently-introduced Protovale CM9 CoverMaster [6]; but in truth, the CM9 is an intermediate stage (for high-tensile steel only) in the development of the "Asdic" cover meter for both high-tensile and stainless steels.

Bar Diameter Dependency

For high-tensile bars, the overall signal received is observed to vary approximately linearly with bar diameter. Because the signal strength varies very rapidly with distance (cover) to the bar, there is only a relatively small variation of indicated cover owing to changes in assumed bar diameter. However, especially for the smaller sizes, the overall signal from stainless bars was observed to vary much more rapidly with bar diameter, and so errors in assumed bar diameter can give rise to significant errors in deduced cover. This implies that the cover to bars of unknown size can not be measured accurately unless a means of determining bar diameter can be found.

Determination of Bar Diameter

On closer inspection, it was discovered that different bar diameters gave rise to eddy currents (and received coil voltages) with markedly different decay time-constants; and it was this effect that accounted for the greatest part of the overall signal variations. The decay of the received signal voltage v as a function of time t can be expressed as

$$v_t = v_0 \, \mathbf{exp} \, (-t/\tau)$$

where τ is the decay time-constant.

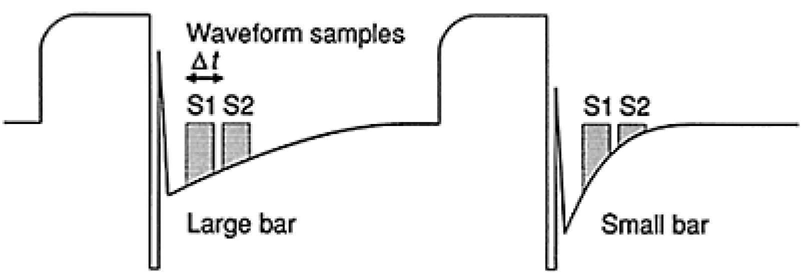

Figure 3 Sampling of receiver waveform to estimate decay time-constant

If two measurements of v are taken at times t and $t+\Delta t$, the time-constant can be derived from the ratio of the two voltages

$$\tau=\Delta t/[\log_e (v_t/v_{t+\Delta t})]$$

The time-constant is known to increase monotonically with bar diameter, but an exact relationship can not be derived and can only be measured experimentally. In practice, it is not necessary to solve for τ, merely to tabulate $v_{t+\Delta t}/v_t$ *versus* diameter. In the solution adopted, both the initial delay and the incremental delay Δt were each nominally 10µs. The method of sampling the waveform is depicted in Figure 3. The ratio of the signals after the two delays was calculated by an 8-bit microprocessor, and the appropriate diameter found from a look-up table.

Steel Grade Determination

In making the experimental measurements of decay time-constants of different sizes of stainless-steel bars, it was also necessary to repeat the measurements on each bar for a range of covers, in order to establish whether the delayed voltage ratio showed any dependency upon bar distance. It was found that there was negligible variation of ratio with cover, so that the expected ratio for each bar size occupied only a narrow range of values, with a wide “dead zone” separating them. However, particularly for the smaller bar sizes, there was a significant difference in the ratio for different steel **grades** (304 and 316), and so in these cases the method can also be used to determine steel grade. Fortunately, the signal **strengths** at any cover were not significantly affected by steel grade, so there is no uncertainty in measurement of cover to a bar of known size but unknown grade.

Experimental measurements of the time-constants of high-tensile bars were also made. The decay of the signal from ferrous bars is not strictly exponential; nevertheless, the ratio of the signal strengths at two different sample delays can still be used to derive an “effective” decay time-constant. In stark contrast to the stainless bars, high-tensile bars of all diameters between 8 and 40 mm exhibited essentially the same value of the $v_{t+\Delta t}/v_t$ ratio: this means that this decay-analysis technique can not be used to determine the **diameter** of high-tensile bars, but it can be used to recognise when bars are high-tensile rather than stainless-steel.

Effect of Neighbouring Bars

All previously-known methods of deducing (high-tensile) bar diameter, by utilising absolute or relative signal strengths, are prone to errors in the presence of neighbouring bars within the magnetic field of influence of the search probe, and almost invariably result in an over-estimate of bar size. However, this new decay-analysis method does **not** exhibit the same shortcoming.

Nearby parallel bars, of the same diameter as the bar being measured, will contribute an additional signal component of arbitrary amplitude but identical time-constant; because only the decay time is analysed, and the absolute amplitude is immaterial, the deduced bar diameter is totally unaffected by these neighbouring bars.

Furthermore, whereas the signals received from high-tensile bars are strongly dependent upon the angular orientation of the search probe with respect to the bar, the orientation effect from stainless-steel bars is much less marked in amplitude and even less noticeable in decay time; as a result, transverse bars of the same size also produce negligible errors in diameter-determination.

If the interfering nearby bars are of a different diameter to the bar being measured, the deduced diameter will lie somewhere between the diameters of the measured and interfering bars, weighted towards whichever bar is yielding the strongest signal. This will mean that if, for example, the interfering bar is two sizes larger (or smaller) than the bar being measured, and is contributing a signal whose amplitude is less than one-quarter of that of the bar being measured, the deduced bar bar diameter will be in error by less than half a bar size; this is sufficient to invalidate the estimation of steel **grade,** but the deduced **diameter** will nevertheless be correct.

A further practical difficulty when attempting to deduce the diameter of high-tensile bars by measurement of signal amplitudes, is the need to position the search probe accurately over and aligned with the bar under test; because this new decay-time analysis does not rely on absolute amplitudes, diameter measurements remain accurate even when made with the head displaced somewhat from the line of the bar, either laterally or angularly.

As far as the author is aware, this new bar-sizing method is the only one to exhibit such a tolerance of positioning errors and such a high degree of immunity to the proximity of nearby bars of similar size.

Combined Stainless/High-tensile Instrument

The circuitry for stainless-steel requires a single wide-band receiver amplifier, and two differential-input integrators to derive the (dc level) signals from the early and late samples. The search head for high-tensile bars contains two sets of coils spaced apart by nominally 20mm in order to implement the “spaced-ratio” method of bar-diameter determination [7], and therefore requires two sample-pulse integrators, one each for the main and auxiliary coils. Both instrumental techniques require pulse generators to generate the transmit and sampling control pulses; these are implemented by counting and decoding master clock pulses. The basic clock period for the stainless-steel version (and hence the width of all pulses derived from it) needs to be half that of the high-tensile version. The challenge was to find a way of implementing the equivalent of two separate instruments in the same housing, without any unnecessary circuit duplication. The approach used was as follows. For high-tensile steel, transmitter pulses are sent to the main lower coils and auxiliary upper coils alternately, but the single receiver circuit is permanently connected to the main coils, thereby ensuring the same amplification factor for both direct and spaced signals; for stainless steel, all transmitter pulses are sent to the main coils. There are two identical integrators, each with associated sampling gates. For high-tensile steel, the “main” sampling gate is activated on those frames in which the main coil transmits; the “auxiliary” gate samples on the alternate pulses, with the same time delay. For stainless steel, sampling pulses are applied to both gates every frame, but the “auxiliary” samples are delayed with respect to the “main” samples. For high-tensile steel, the master clock is 48kHz and the sample delay is nominally 20μs; for stainless steel, the master clock is 96kHz and the main and auxiliary sample delays are nominally 10 and 20μs respectively. The outputs of the two integrators are continuously-present dc levels; the analogue-to-digital converter digitises these alternately at a rate which is independent of the above timings. In this way, there is no duplication of signal-processing circuitry; only one additional integrated circuit package is required to demultiplex the different sampling pulses.

GUIDELINES FOR VARIOUS TYPES OF REINFORCEMENT

"Conventional" High-Tensile or Mild Steel

For location, use any rebar locator or cover meter that has a directional search probe and is known to be capable of resolving bars at the pitches anticipated within the structure. For cover measurement, use a cover meter which satisfies all the requirements above for location, and can also be shown to give correct readings at the anticipated levels of bar congestion within the structure.

For diameter estimation, all requirements in the preceding two paragraphs must be met, and in addition, the chosen instrument should first be trialled on a known structure similar to the structure to be tested.

Either of the two instruments described here may also be used; but if the "Solution A" instrument is set for stainless steel, it will under-estimate cover; "Solution B" would similarly under-estimate cover, but should recognise that the reinforcement is ferrous rather than stainless, and therefore indicate true cover and diameter.

Epoxy-Coated Steel

The epoxy coating should have no influence on the measurement whatsoever; therefore follow the guidelines for high-tensile steel in the section above.

Galvanized or Plated Steel

Owing to the low excitation frequencies used, and the negligible fraction of the total mass of metal that is represented by the coating or plating, the reinforcement will respond in the same way as conventional ferrous reinforcement; therefore follow the guidelines for high-tensile steel in the section above.

Stainless Steel

For location and cover measurement and diameter determination, it will be necessary to use "Solution B" as it is the only method known to fulfil all requirements.

For location and cover measurement of bars of known diameter, "Solution A" is also suitable.

If either of these instruments is set to high-tensile mode, they will (possibly grossly) over-estimate cover, and may fail to even locate the smallest sizes of stainless bar. For location only, the Imp Stainless-steel Wall-tie Locator is also suitable. Instruments that have not been designed specifically for stainless steel will not be suitable.

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STRUCTURAL DESIGN USING EPOXY COATED REINFORCEMENT

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ABSTRACT. Fusion bonded epoxy coated reinforcement (FBECR) has been developed to help combat problems of corrosion in reinforced concrete structures. The surface texture of the coating is smoother than the normal mill scale surface of reinforcing bars and alters bond characteristics of the bar. Although FBECR has now been in use for more than 30 years, rules for design using the material are not well developed. The purpose of this paper is to review available research and to derive recommendations for design practice which to enable structures reinforced with FBECR to achieve equivalent performance to that of structures reinforced with millscale surface ribbed bars. The paper shows that existing Code rules for anchorage strength of ribbed bars at the ultimate limit state offer a satisfactory margin of strength, but that modifications are required when assessing serviceability performance.

Keywords: Reinforcement, Epoxy Coated, Bond, Anchorage, Lapped Joint, Crack Control, Deflection.

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INTRODUCTION

Fusion bonded epoxy coated reinforcement (FBECR) was developed in the USA in the 1960’s in response to widespread problems of reinforcement corrosion in concrete bridge decks. The epoxy coating is impermeable and an electrical insulator, and resists corrosion of reinforcement by providing a physical barrier to the oxygen and moisture necessary for the corrosion reaction, and by interrupting electrical continuity of the corrosion cell. The USA was the first country to conduct a major study into the effectiveness of FBECR and the first to introduce a national standard for the material[1]. By 1987, epoxy-coated bars accounted for around 5% of total reinforcement consumption in the USA. In the UK, a dedicated plant for production of FBECR was set up in 1987. FBECR is now also produced in several other European countries, in Japan and in the Middle East. However, although the product has now been available for some time, it has yet to be widely assimilated into Codes of Practice for design.

The surface texture of an epoxy coating is much smoother than the normal millscale surface of a hot rolled, or 'black' reinforcing bar, and affects the interaction, or bond, between reinforcement and concrete. For design purposes, bond is considered to be a factor in both serviceability and ultimate strength. At the serviceability limit state, crack widths and deflection are influenced. At the ultimate limit state, strength of laps and anchorages depends on bond. In addition, bond characteristics of flexural reinforcement will influence rotation capacity of plastic hinges. Extreme loss of bond is also known to influence flexural and shear capacity of beams. However, changes of this degree lie beyond the change in bond that will arise from epoxy coating, and is not considered further here.

Aim

This paper reviews data on structural performance of epoxy coated reinforcement, with the aim of deriving recommendations for design practice to enable structures reinforced with FBECR to achieve equivalent performance to that of structures reinforced with millscale surface ribbed bars.

GENERAL REVIEW OF BOND OF FBECR

It is widely accepted that a coating thickness of 0.2mm provides an optimum compromise between durability and structural performance requirements, and this is reflected in product Standards[1,2]. Results reviewed in this paper are confined to coating thicknesses of this order applied by spray methods. Investigations into bond of FBECR may be roughly subdivided into two main categories, namely investigations concerned with bond itself, and investigations concerned with the influence of bond on structural performance. Investigations in the former category use specialised bond test specimens. Axial pullout and beam end type specimens are widely used. The effect of bond on structural performance is examined using test specimens more representative of entire structural components such as beams, slabs or columns.

Assessment of performance in specialised bond tests has been based on a variety of criteria, including failure load, limiting slip at a specified bond stress, and on bond stress developed at a specified slip, both at free and at loaded ends of test bars. In all cases, bond strength of coated bars was compared with that of similar black bars in an identical specimen. However, bond stresses from pullout type tests are often of little direct relevance to design[3]. The variety of criteria used and the general absence of established correlations between bond strength as assessed in specialised bond tests and performance in a structural element does not make it easy to determine a 'bond reduction factor' for coated bars appropriate to practical design. Cairns & Abdullah report that the ratio of bond stress developed by an epoxy coated bar to that developed by a nominally identical uncoated bar varies from 0 to 1.1, depending on the slip at which comparison is made,

Figure 1[4]. The dependence of bond reduction on slip suggests that the search for a unique bond reduction factor for coated bar will be fruitless, and that

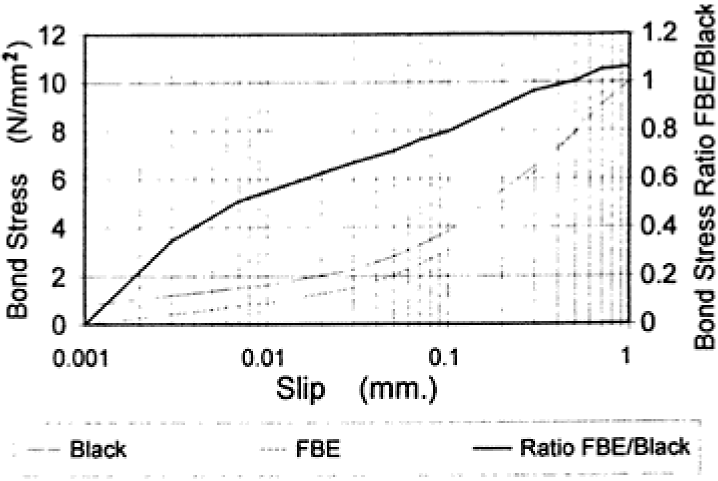

Figure 1. Variation of bond stress with slip for black and epoxy coated reinforcement[4].

different reduction factors may be appropriate to different aspects of bond performance. Before considering what factor(s) may be suitable for design, a review of the principal factors to influence the relative bond performance of FBECR and black bars will be presented

Confinement

Bond action of ribbed bars generates bursting stresses that tend to split the surrounding concrete. It is established that bond strength increases with confinement to the bars, either from increased cover, transverse reinforcement, or lateral pressure. Depending on the Code of Practice in use, some or all of these factors may be taken into account in detailing of laps and anchorages. Choi et al demonstrated that the absolute increase in bond strength with increasing cover is similar for both coated and black bars, and hence that the difference, when expressed as a ratio, reduces with increasing cover[5]. Treece and Jirsa arrived at a similar conclusion from tests on lapped joints[6].

Casting position

Compaction of concrete tends to be poorer close to the top of a pour, and settlement of fluid concrete may lead to formation of voids below bars cast near the top of a pour. Bond strength is dependent on the quality and compaction of the surrounding concrete, and bars cast in locations near the top of a pour tend to be weaker in bond than those cast near the bottom. In an extensive study employing beam end test specimens, Hadje-Ghaffri et al examined the interaction of slump, casting position and compaction[7]. They found bond strength of coated bars to be affected less by 'poor' casting position than black bars, and suggested a reduced top bar factor. Tests conducted at BCA[8] also showed that a'poor' casting position affected epoxy coated bars less than black bars.

Rib Geometry

In tests on specimens representing lapped joints Cairns and Abdullah showed that the strongest epoxy coated bars could develop nearly the same bond strength as the weakest black bars[9], and that the weaker rib patterns, with lower relative rib areas, generally exhibited smaller bond strength reductions. Choi et al[5] also found a strong dependence of bond strength reduction on rib pattern, but reported that bars with a higher relative rib area were less affected by coating. The difference may be associated with the provision of confining reinforcement, or with a correlation between relative rib area and bar diameter in Choi's study. The opposing conclusions of these two studies have still to be accounted for.

Creep and Cyclic Loading

Clifton and Mathey report that creep of epoxy coated bars is of the same order as that of black bars at service stresses[10]. Similarly, Johnston and Zia[11] concluded that behaviour of coated bars under cyclic loading was essentially similar to that of black bars, although a tendency for the differential to diminish as the number of cycles increased was noted. In a series of splice tests, Cleary & Ramirez also found repeated loading to be less detrimental to coated bars[12].

Hooks and bends

Hamada et al a report that anchorage capacity of hooks and bends are reduced by coating, although the differential between coated and black bars is slightly less than for straight bar anchorages[13].

Fire resistance

Fusion bonded epoxies are thermosetting polymers that cannot be turned to a molten state by application of heat, although they may pyrolyse at higher temperatures. The material does soften around the glass transition temperature of approximately 110°C, however. Pullout tests conducted on coated and black reinforcement cast into a reinforced concrete slab[14] under fire conditions showed that all test bars were able to attain their yield strength. Although it is evident that the coating material softened at temperatures above 110°C and that bond behaviour was affected, it should not be concluded that fire resistance is lessened by use of epoxy coated bars. It is also necessary to bear in mind reservations concerning the relevance of pullout type tests to practical situations set out above.

Summary

It is generally agreed that bond strength and stiffness of coated bars is lower than that of 'black' bars. The strength reduction is generally estimated at between 0% and 25%, although reductions of around 40% have been reported. An assessment of bond based on performance of straight bars at ambient temperatures under monotonic short term loading under conditions of minimum confinement consistent with Code provisions should give results that will be conservative for other circumstances.

DESIGN AND DETAILING WITH FBECR: GENERAL CONSIDERATIONS

Coated bars have now been produced for around 30 years, but it is only relatively recently that Codes of Practice have introduced guidance on design and detailing of concrete structures reinforced with epoxy coated bars. Reduction factors for bond strength at anchorages and lapped joints are now included in a few national Codes of Practice[15,16], although the accuracy of the factors introduced is debated. At the serviceability limit state, an epoxy coating is either taken to have no effect on serviceability behaviour[17], or any effect is ignored. Other Standards documents, such as EC2[18] and the CEB-FIP Model Code[19], recognise the influence of bond on serviceability performance, but do not provide specific values for use when detailing with FBECR.

Many studies on bond of FBECR have been concerned primarily with derivation of a numerical factor to quantify the bond strength reduction, crack width increase, or other change in structural performance attributable to coating. Such factors can be obtained through direct comparison of measurements of performance of nominally identical specimens reinforced with black bar and with FBECR. This approach has two major disadvantages:

a) the relationship between structural performance of FBECR and black bars is dependent on the design of the test specimen. Extrapolation to other circumstances may be misleading.
b) the conditions under which FBECR and black bars are used in practice may differ, and could offset or accentuate differences attributable to bond.

It is thus necessary to examine the theoretical and practical background to the simplified rules presented in Design Standards and Codes to derive recommendations for design.

SERVICEABILITY LIMIT STATE: CRACKING & DEFLECTIONS

For practical design, crack widths and deflections are controlled through simple ‘deemed to satisfy’ rules, and not by direct calculation. This is at least in part because serviceability is intrinsically less critical than ultimate the ultimate limit states, and the major calculation effort required for a detailed estimate is rarely justified. Crack widths are controlled by limiting transverse spacing between longitudinal bars, and deflections controlled by limiting the span/depth ratio. Bond characteristics do not feature in such rules. These ‘deemed to satisfy’ rules are not considered in this paper, which instead concentrates on adaptation of the rules provided for more detailed assessment of performance at the serviceability limit state. However, the approach evolved could subsequently be used in parametric studies for the development of simplified rules.

In a cracked cross section tension forces are carried by the steel only. However, concrete does possess some tensile strength, and between transverse cracks may carry part of the tension force. Bond characteristics influence the rate at which load is transferred between bar and concrete, and influences structural performance in two ways:

a) Between transverse cracks, reinforcement strains are reduced below the value that would be calculated if the section were fully cracked, Figure 2. The reduction in average bar strain is termed tension stiffening, and is dependant on bond.
b) The distance between successive transverse cracks is influenced by the rate at which tension in the longitudinal bars is transferred to the surrounding concrete. The stiffer the load/slip relationship, the more rapid the transfer of force, and the shorter the distance required to increase tensile stress in the concrete to the level at which a crack will form, Figure 2.

Deflection

Deflection of a beam may be expressed by Equation 1, which shows that a reduction in tension stiffening will result in increased deflection 6. The magnitude of ε_{ts} depends not only on bond characteristics, but also on the proportion of reinforcement in the section, the cross sectional shape, the load level at which the comparison is made, the grade of concrete used, and on whether short or long term behaviour is considered. The relationship between deflections of beams reinforced with coated and black bars therefore cannot be expressed by a unique coefficient.

$$\delta = \int\int 1/R.dl = \int\int \varepsilon_m/(d-x).dl = \int\int (\varepsilon_{crt} - \varepsilon_{ts})/(d-x).dl$$

Equation 1

where 1/R—curvature of section
1—distance along beam
ε_m—mean tensile strain in longitudinal reinforcement
(d-x)—distance from neutral surface to centroid of tension reinforcement
ε_{cr}—strain in reinforcement calculated on basis of fully cracked section
ε_{ts}—tension stiffening strain

Several investigators report little change in deflection where coated bars are used. Closer inspection reveals that most of the specimens were relatively highly reinforced, and would be insensitive to loss of tension stiffening. The presence of a lapped joint may also have masked loss of tension stiffening in a number of studies. In a series of tests on seven pairs of nominally identical beams of various cross sections, Cairns measured an average increase in deflection of around 20% where coated bars were used. Deflections for individual pairs differed by between 5% and 35%[20]. Taken as an average, the increased deflections corresponded to a reduction in bond of around 50% for coated bars. The

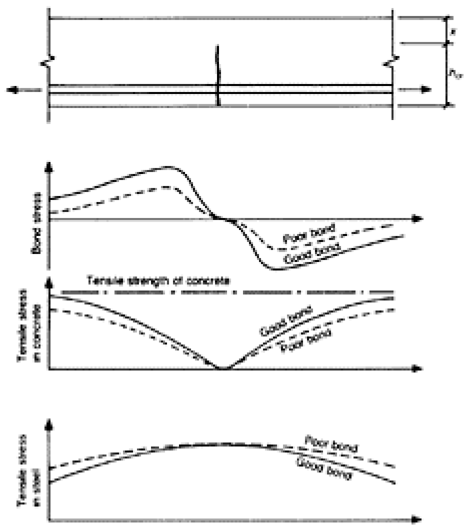

Figure 2 Influence of bond on cracking and deflection

relationship is consistent with the difference in bond stresses at slips corresponding to serviceability limit state conditions, measured in RILEM pullout tests, and indicate that crack control performance of FBECR is equivalent to that of plain round (uncoated) bars. This change can readily be introduced within the detailed procedures of EC2 for calculation of beam deformations by setting bond coefficient β_1 to 0.5, as for plain round bars.

Crack Widths

For practical purposes, mean crack width w_m may be estimated as mean strain in reinforcement ε_m times the distance S_{rm} between cracks, Equation 2. A variety of of expressions have been proposed for estimating crack spacing and are generally similar in form, Equation 3. Coefficient k1 takes account of bond properties, and is greater for bars with less stiff load-slip behaviour. The increase in k_1 is widely assumed to be inversely related to bond. A change in bond characteristics may thus have a twofold influence on crack widths, as crack spacing will be greater and tension stiffening less when bond stiffness is reduced.

$$w_m = S_{rm}\,\epsilon_m$$

Equation 2

$$S_{rm} = k_c.c + k_1\,k_2\,d_b/\varrho$$

Equation 3

where k_c k_1 k_2 are coefficients dependent on probability of excedence, on bond characteristics, and on pattern of strain in the section respectively,

c—minimum cover

ϱ—geometric ratio of reinforcement to surrounding concrete

Several Authors report increased crack spacing and crack widths when FBECR is used in place of black bars in nominally identical beams. Increases of up to 100% have been reported. From an analysis of crack measurements on seven pairs of beams of different cross section, in which one of each pair was reinforced with epoxy coated bars and the other with identical black bars, Cairns deduced that bond stiffness of epoxy coated bars was half that of black bars[23]. In Figure 3, the analysis is extended to results reported in other studies. Tension stiffening has been assumed to be halved when coated bars are used, and coefficient k_1 (Equation 3) doubled. The number of cracks measured in each study is

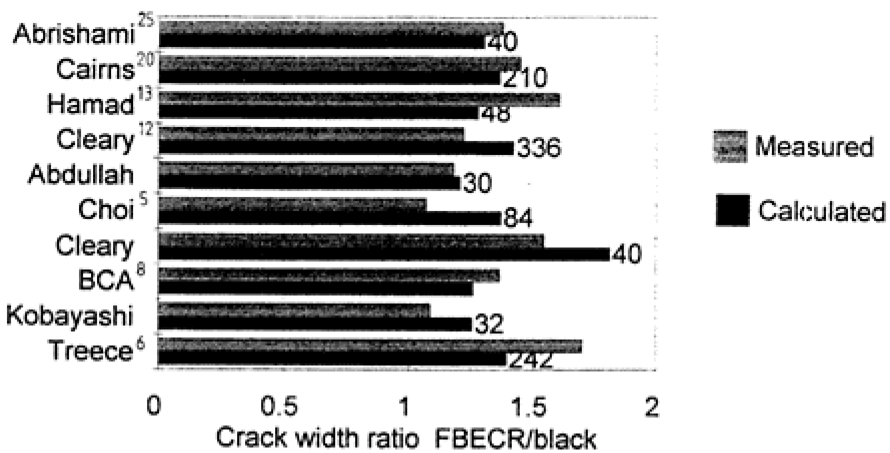

Figure 3. Comparison of crack widths for specimens reinforced with black and epoxy coated bars.

noted on the graph. Predictions are generally in good agreement with observations. However, some tuning of the approach may be required for inclusion in existing Codes. For example, EC2 modifies Equation 3 by substituting 50mm in place of the variable cover, and a better fit to test data is obtained if k_1 is increased by 40% instead of 100%.

ULTIMATE STRENGTH OF LAPPED JOINTS

Codes of Practice make the assumption that bond stress is uniform throughout the length of a lapped joint. This leads to the erroneous (but convenient) assumption that lap strength is directly proportional to lap length. Statistical analyses of test data for black bars provide ample evidence that this is not so, and that the increase in strength is less than proportional to the increase in lap length. Bond tests are generally carried out with lap lengths short enough to ensure the occurrence of bond failure before lapped bars start to yield. However, Code values for design bond strength must be based on the lap length required to develop characteristic strength of the reinforcement. Figure 4 illustrates the variation in strength of lapped joints of black and epoxy coated bars measured in three studies[8,21,22]. Slopes of paired plots are similar, but show that the reduction in average bond strength with increasing lap length is less marked for epoxy coated bars. Results from shorter laps will therefore overestimate how much full strength laps are weakened by coating.

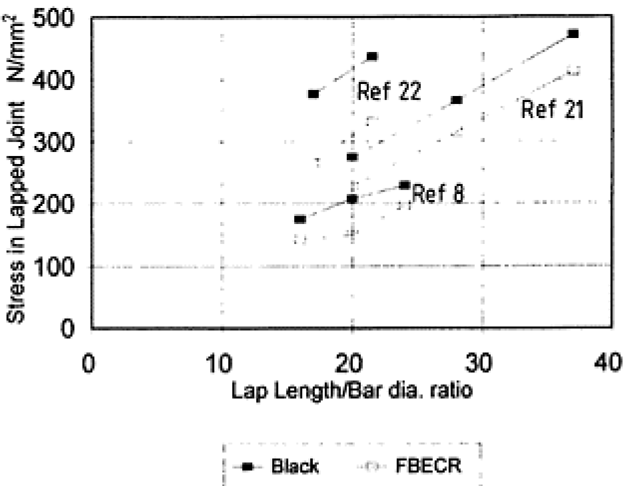

Figure 4. Variation in strength of lapped joint with lap length.

Bond strength in a splitting failure mode is considered to be limited by the resistance of a concrete hoop around the bar, together with additional resistance from any confining reinforcement crossing the failure plane, to splitting. Tensile capacity of the concrete hoop is assumed to be dependant on cover thickness or clear spacing between adjacent lapped joints. Design bond strengths are based on minimum values of cover, spacing and confining reinforcement. A minimum cover of 1 times bar diameter is generally specified for structural reasons, notwithstanding durability and fire resistance requirements[18]. Requirements for reinforcement to confine lapped joints are also imposed. These detailing requirements define “least favourable” conditions for which design bond strengths for black bars are derived. Design bond strengths will be conservative to some degree for covers and bar spacings in excess of these minimum requirements.

FBECR is to be considered to provide an additional measure of corrosion protection for reinforcement, and it is not suggested that coated reinforcement will enable reduced cover for durability. As the extra cost of FBECR is generally justified only where there is significant risk of reinforcement corrosion, its use will be primarily in harsh environments, and it will not generally be specified for exposures less severe than class 3, ‘Humid with frost and de-icing salts’, as defined by EC2, for example. Concrete cover to reinforcement in structures subjected to such environments will be relatively high, and will result in cover/bar diameter ratios of at least 1.5 even for 32mm diameter bars. The conditions under which black bars are used are thus less favourable to bond than the conditions under which FBECR is used. Semi-empirical equations such as that developed by Orangun, Jirsa and Breen[23] show that an increase in cover ratio from 1.0 to 1.5 will produce an increase of 25%-30% in bond strength. The loss attributable to coating will thus be largely offset by a gain from the additional cover.

Figure 5 compares measured strength of lapped joints of epoxy coated bars with that calculated using design bond strengths in EC2 for black ribbed bars. Results for both top and bottom cast bars are represented. Filled markers represent results for specimens complying with detailing requirements of EC2. However, many specimens reported do not comply with EC2 detailing requirements, either because bar spacing was below the permissible minimum, or because confining reinforcement provided was insufficient, and results are denoted by empty markers. Although results show a wide scatter, most measured strengths exceeded those predicted by EC2 by a factor exceeding the partial safety coefficient on bond of 1.5. (Scatter in results for coated bars is no greater than that for black bars). Figure 5 therefore shows that current EC2 provisions for lapped joints of black bars also offer an acceptable margin of safety for coated reinforcement, although the wide scatter also suggests that accuracy of the rules could be improved.

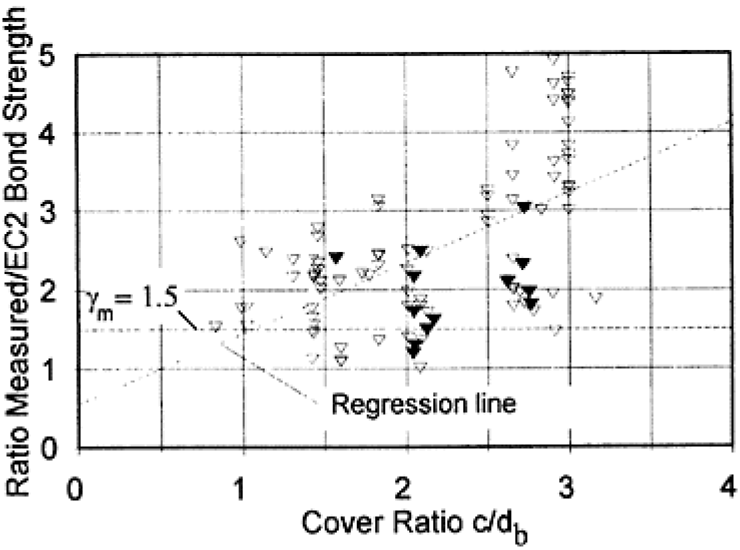

Figure 5. Comparison of measured bond strength with EC2 design values.

Plastic rotation capacity

Plastic rotation capacity is important when redistributing elastic moments through the formation of plastic hinges. Rotation capacity increases with the length of and the mean reinforcement strain within the plastic zone. Mean strain is influenced by tension stiffening, as discussed earlier in connection with serviceability limit state performance, and will be greater for bars with a less stiff bond-slip relationship[24]. It is therefore expected that plastic rotation capacity will increase when epoxy coated bars are used. Little data exists on ductility of beams reinforced with epoxy coated reinforcement. However, Abrishami et al reported a small reduction in ductility of a beam with 0.9% reinforcement when epoxy coated bars were substituted for black bars[25].

CONCLUSIONS

It has not been possible to fully explore all aspects of the bond performance of FBECR in a paper of this nature. The following conclusions, of relevance to Codes of Practice, are nonetheless warranted.

1 Bond stiffness and strength is reduced by an epoxy coating.
2 Crack control performance of FBECR is equivalent to that of plain round bars, and beams reinforced with FBECR exhibit greater deflections than similar beams reinforced with black bars. The difference in structural performance is dependant on the properties of the member and on the loading as well as on bond.

3 Ultimate strength of laps and anchorages is reduced by an epoxy coating. For practical design purposes, this reduction is offset by an increase in bond strength where higher cover is used for durability in adverse exposures. A comparison with provisions of EC2 suggests that present rules for design of lapped joints of black bars provide an adequate margin of safety against failure of lapped joints of coated bars. Validity of this conclusion is at present restricted to bars of up to size 25 in beams or slabs constructed of concretes not exceeding Grade 40, and detailed for chloride contaminated exposures.

Further Work

1 There is no data on crack control properties of FBECR in members loaded in direct tension.
2 The reduction in tension stiffening associated with FBECR has not been adequately quantified.
3 It would be of value to measure strength of lapped joints and of anchorages of black and coated bars under respective 'least favourable' conditions of use to verify that Code rules provide a consistent margin of safety.
4 More data is required on the effect of coating on plastic rotation capacity.
5 Further work to determine the influence of rib geometry on the reduction in bond strength attributable to coating is desirable.

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SUPERCOVER CONCRETE: A NEW METHOD FOR PREVENTING REINFORCEMENT CORROSION IN CONCRETE STRUCTURES USING GFRP REBARS

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ABSTRACT. The paper describes a new design method called *Supercover Concrete* for preventing reinforcement corrosion in concrete structures. It involves providing nominal covers to the steel reinforcement of 100mm and introducing glass fibre reinforced plastic (GFRP) rebars in the cover zone to control surface crack widths.

Short term loading tests on single span, simply supported reinforced concrete beams made using this design show that GFRP rebar are effective in reducing surface crack widths to values recommended in Eurocode 2 and BS8110. Moreover, the structural strength and the deflections which occur are not adversely affected. Results of preliminary tests to determine the long term performance of this design are also reported.

Keywords: Glass fibre reinforced plastic (GFRP) reinforcement, Corrosion, Prevention, Supercover Concrete.

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INTRODUCTION

Vast sums of money are spent annually in the U.K. and elsewhere on the repair and maintenance of concrete structures [1]. A large proportion of this expenditure is to deal with the deterioration caused by reinforcement corrosion. This corrosion is attributable to carbonation of the concrete and/or the presence of chlorides in solution around the reinforcement.

Many ways have been suggested for preventing this problem including treating the surface of the concrete with silanes or siloxanes [2], replacing a proportion of the cement with ground granulated blast furnace slag [3] or pulverised fuel ash [4], and using stainless steel or coated reinforcement. A cheaper alternative is simply to increase the depth of cover to the reinforcement.

Although easy to implement, deep covers will give rise to cracks with large widths at the concrete surface. These may be undesirable for reasons of durability but are often unacceptable because of appearance. However if rods made of non-metallic materials such as glass fibre reinforced plastic (GFRP) were to be introduced in the cover zone, it should be possible to control surface cracking to within acceptable limits. Using this approach, the steel reinforcement would primarily provide the tensile strength and will not corrode for the life of the structure due to the deep concrete cover. The non-metallic reinforcement situated near to the concrete surface would primarily control surface crack widths and will also not corrode.

If such a system could be developed, it would offer the following advantages:

(1) low cost and cheap to implement solution
(2) primarily relies on existing materials and technology
(3) applicable to a wide range of structures.

The purpose of this paper is to provide further details of this method, termed Supercover Concrete, and to describe some of the laboratory investigations currently being undertaken to assess its performance.

SUPERCOVER CONCRETE

The main feature of this method is the reinforced cover. Two questions that need to be addressed are:

1) what thickness of concrete cover to the steel reinforcement should be provided and
2) is GFRP effective in controlling cracking in the short and long term?

With regards to the first question, it is well established that the penetration of chlorides into hardened concrete roughly follows a square root time function [5]. Moreover, field experience shows that many concrete highway structures in the U.K. show signs of deterioration within twenty years of construction [6]. Thus, if the cover to the reinforcement was increased from 40mm to, say, 100mm i.e. an increase of 60mm above the nominal cover specified in Eurocode 2 [7], the time taken for chlorides to reach the level of the reinforcement will increase by a factor of $(100/40)^2$ i.e. approximately six fold. Therefore providing nominal covers of 100mm to the steel reinforcement should eliminate the possibility of corrosion occurring during the design life of most structures.

The second question is more difficult to answer. Short term flexural tests on concrete beams reinforced with equal amounts of steel or GFRP show that the beams reinforced with GFRP experience greater deflections and have larger surface crack widths compared with steel reinforced beams [8]. This is due to the fact that the stiffness of GFRP rebars is about 25% that of steel and that the bond strength of GFRP with concrete is also significantly lower being approximately 65 % that of the steel bars to concrete strength [9]. The long term performance is also difficult to judge since there is evidence that certain glass fibre composites deteriorate in alkaline environments typical of those found in concrete [10, 11]. In addition, most resins are reported to have poor resistance to creep [12]. Both factors could therefore increase the surface crack widths with time.

Since it was impossible to predict the behaviour of the GFRP rebars in this design it was considered necessary to carry out laboratory investigations. The following section describes the work which is currently under way to assess both the short term and long term behaviour of beams made with this design.

EXPERIMENTAL PROCEDURE

Short term loading tests

Four reinforced concrete beams, 300mm wide, 400mm deep and 3100mm long, were cast in timber moulds. Fig. 1 shows the reinforcement details. Mix details are given in Table 1. Two batches, each consisting of 500 kg of concrete, were used to make each beam. An electric vibrator was used to compact the concrete. Three concrete cubes, 100mmx100mm x100mm, were cast from each batch of concrete to measure the compressive strength.

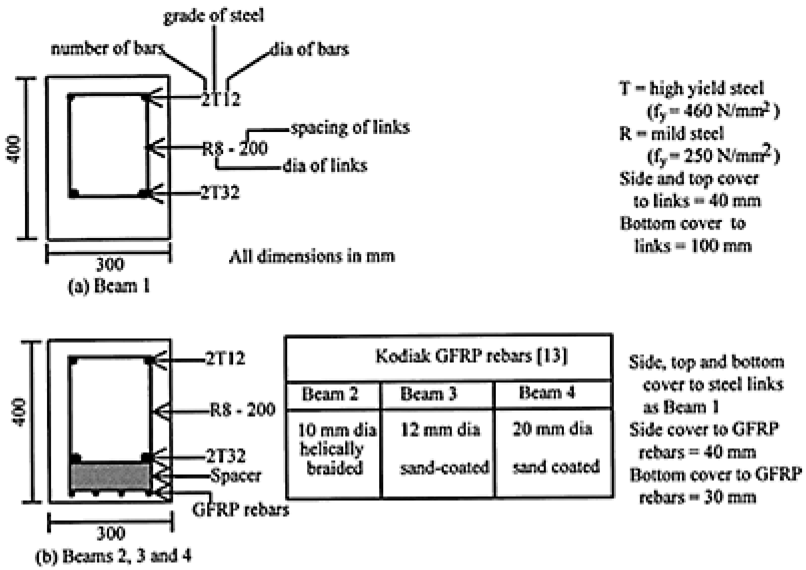

Figure 1 Beams 1–4—Reinforcement details

Table 1 Beams 1–4—Mix proportions (by weight)

OPC	1.00
Sand (Zone 3)	1.78
10mm aggregate (flint gravels)	1.31
20mm aggregate (flint gravels)	1.96
Water	0.43

The beams were allowed to cure in their moulds for two days. Thereafter they were removed from the mould and wrapped in damp hessian and polythene, and allowed to cure for a further 26 days.

Prior to testing, the beams were painted white in order to ease monitoring of crack development during loading. The beams were tested in bending using a four point loading arrangement as shown in Fig. 2. The effective span of the beam was 2.9m. The load was applied at a rate of 0.03 kN/sec, generally in increments of 12.5 kN or 25 kN up to a maximum of 210.5 kN in order to produce a design moment of 100 kNm in the 1m central pure bending region of the beam.

At each load increment, the crack widths were measured using a portable microscope. The position and extent of individual cracks was also recorded. In addition, the mid-span defections of the beams were measured by means of a linear displacement transducer.

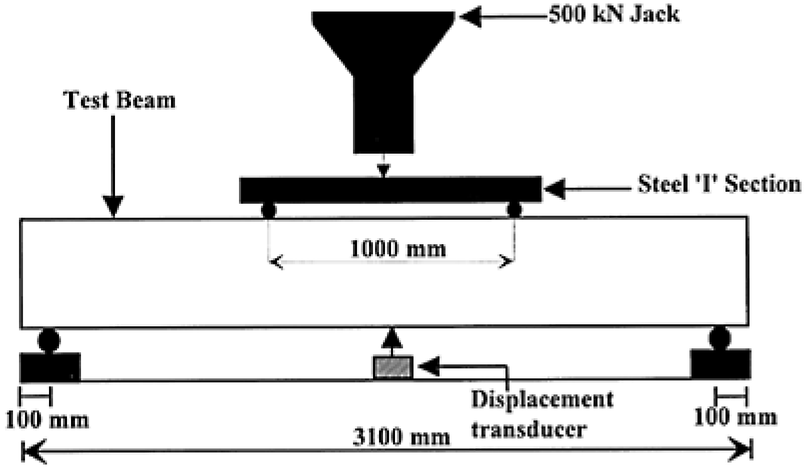

Figure 2 Beams 1–4—Bending test

Long term loading tests

Two identical concrete beams (L1 and L2), 90mm wide, 160mm deep and 1600mm long, were cast in timber moulds. Fig. 3 shows the reinforcement details. The mix proportions are shown in Table 2. The beams were cured for 28 days as previously described for beams 1–4 and painted white prior to testing.

Table 2 Beams L1 & L2—Mix proportions (by weight)

OPC	1.00
Sand (Zone 3)	1.78
10mm aggregate (flint gravels)	3.27
Water	0.43

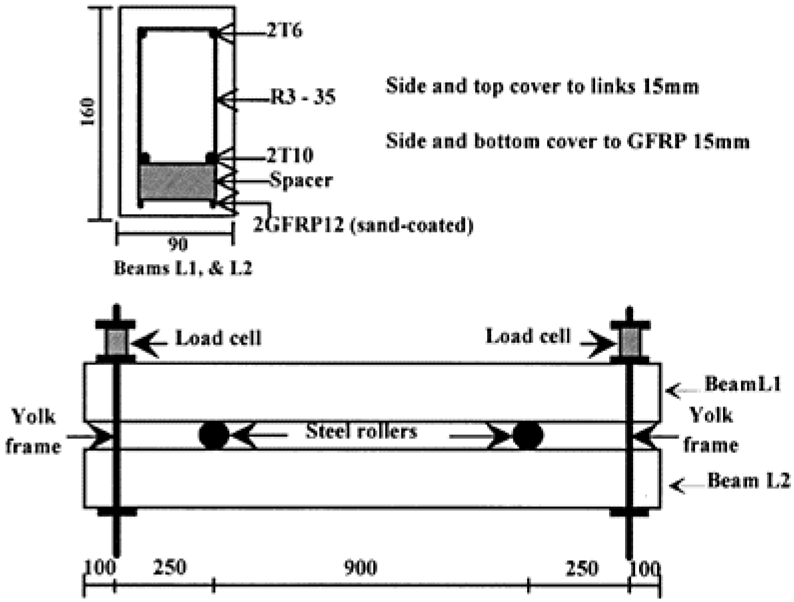

Figure 3 Beams L1 & L2—Reinforcement details and testing set-up

The beams were stressed in four point bending using steel rollers and yoke frames as shown in Fig. 3 to induce cracking in the GFRP reinforced covers. The loading at each yoke was increased to 16kN to produce a design moment of 4kNm in the 900mm pure bending regions in the beams. Any relaxation in the loading due to creep effects was corrected by monitoring the load cells attached to the steel yokes. The beams were stored in a room at a temperature and relative humidity of, respectively, 21±4°C and 35±10%.

The mid-span deflections of the beams were measured using dial gauges and the crack widths were monitored using a portable microscope.

RESULTS AND DISCUSSION

Concrete compressive strengths

Table 3 summarises the average 28 day compressive strengths for the concrete used in Beams 1–4 and L1 & L2.

Table 3 Average 28 day compressive strengths

	Beam 1	Beam 2	Beam 3	Beam 4	Beams L1 & L2
Compressive strength (N/mm^2)	47	53	53	52	50

Short term loading tests

(1) Crack widths

Fig 4 shows the pattern of cracking and maximum surface crack widths obtained on Beams 1–4 at a load of 210.5 kN. Beam 1, containing only steel reinforcement, had a total of six cracks, four of which exceeded a surface width of 0.3mm. The maximum crack width on Beam 1 was 0.5mm. Beam 2, containing four 10mm diameter braided GFRP rebars, had a total of seven cracks, one of which exceeded a surface crack width of 0.3mm. The maximum crack width on Beam 2 was 0.4mm. Beam 3, with four 12mm diameter sand coated GFRP rebars, had a total of eleven cracks and the maximum crack width was 0.3mm. Beam 4, containing four 20mm diameter sand coated GFRP rebars, had a total of six cracks and the maximum crack width was 0.25mm.

Beams 3 and 4 complied with BS8110 [14] as none of the crack widths exceeded 0.3mm. Beam 2 also complied with BS8110 as more than 80%, actually 85%, of the cracks had a surface width of 0.3mm or less. Beam 1, on the other hand, fell outside the recommendations for crack control in BS8110.

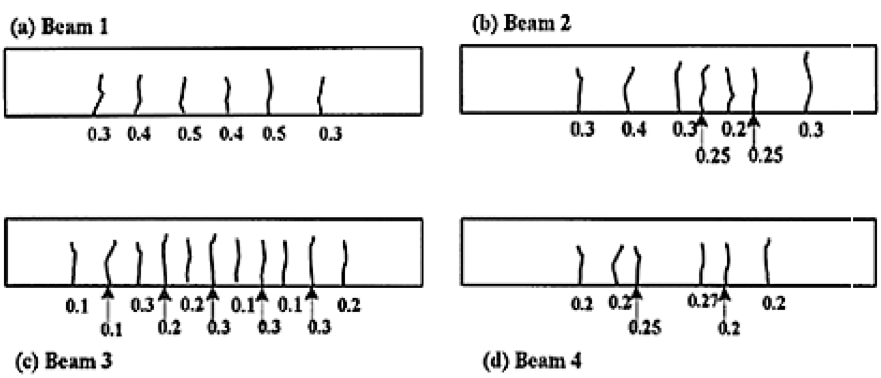

Figure 4 Crack patterns and maximum surface crack widths at a load of 210.5 kN

Fig. 5 shows the predicted and experimental crack widths as a function of design moment for Beams 1–4. The predicted surface crack widths, w, were calculated using equation 1, taken from BS8110 [14]:

$$w = \frac{3a_{cr}\,\epsilon_m}{1 + 2\left(\dfrac{a_{cr} - c_{\min}}{h - x}\right)} \tag{1}$$

where a_{cr} is the distance from the point considered to the surface of the nearest longitudinal bar; ϵ_m the average strain at the level where cracking is being considered; c_{min} is the minimum cover to the tension steel/GFRP rebar; h is the overall depth and x is the depth of the neutral axis.

As can be seen from Fig. 5a, the predicted crack widths for Beam 1 always exceed the experimental values. Generally, there is good agreement between the experimental and predicted crack widths. With Beams 2–4, it was found that if the presence of the GFRP rebars was ignored in calculating the depth of neutral axis and stress in the steel, good agreement existed between the predicted and experimental crack widths (Figs. 5b–5d). This suggests that the GFRP rebars do not significantly affect the moment capacity of the section, which might otherwise result in an over-reinforced failure. The GFRP rebars are largely ineffective structurally probably because they are not tied to the shear reinforcement.

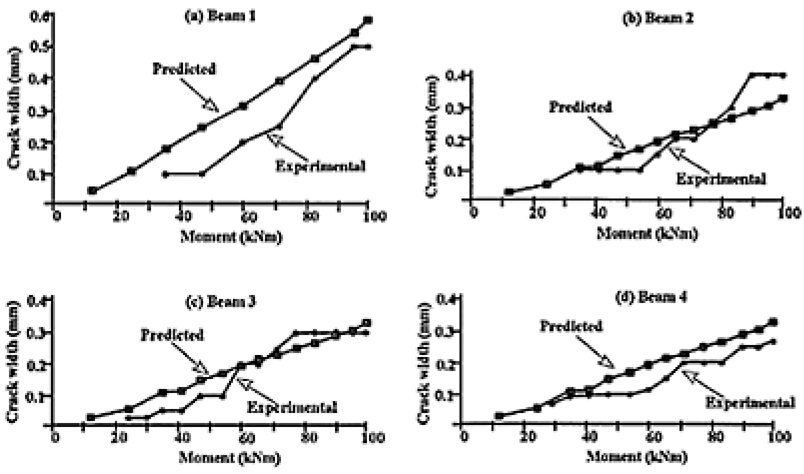

Figure 5 Predicted and experimental crack widths

(2) Deflections

The maximum mid-span defections for Beams 1–4 were, respectively, 4.7mm, 5.3mm, 5mm and 4.0mm. The predicted deflection for Beam 1 was 5.9mm and was calculated using equations 2 and 3, taken from BS8110 [14]:

$$a = K L^2 \frac{1}{r_b} \tag{2}$$

where L is the effective span of the member; K is a constant that depends on the shape of the bending moment diagram; $1/r_b$ is the curvature at mid-span and is given by:

$$\frac{1}{r_b} = \frac{f_c}{xE_c} = \frac{f_s}{(d - x)E_s} \tag{3}$$

in which f_c is the design service stress in the concrete; E_c is the short term modulus of the concrete; f_s is the estimated design service stress in tension reinforcement; d is the effective depth of the section; x is the depth of the neutral axis and E_s is the modulus of elasticity of the reinforcement.

The maximum permissable deflection is 11.6mm and therefore all the beams comply with the deflection requirements in BS8110. Fig. 6 compares the deflections occurring in Beams 1–4. Generally, it can be seen that the sections containing GFRP rebars showed similar deflection characteristics to Beam 1 with no GFRP and therefore the existing approach for estimating deflection in BS 8110 can be used, if the presence of the GFRP is ignored.

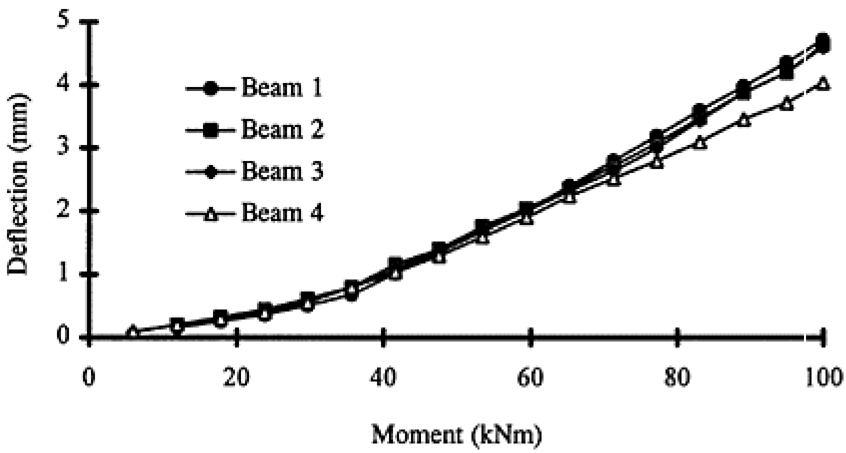

Figure 6 Deflections in Beams 1–4

Long term loading tests

(1) Crack widths

Fig 7 shows the change in maximum surface crack width with time for beams L1 and L2. After five months of testing, the maximum crack widths in beams L1 and L2 were 0.3mm and 0.28mm respectively. The predicted crack width was 0.3mm.

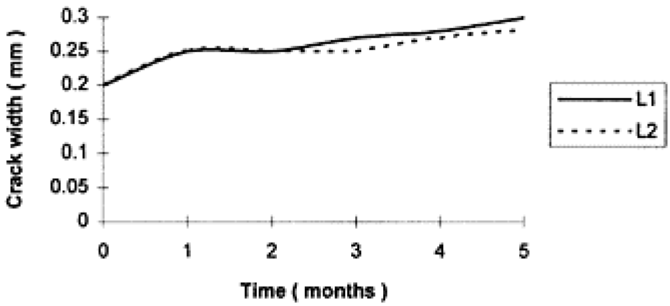

Figure 7 Maximum surface crack widths on Beams L1 & L2

(2) Deflections

Fig 8 shows the change in mid-span deflection with time for beams L1 and L2. After five months of testing the actual deflections in beams L1 and L2 were, respectively, 1.99mm and 2.32mm which are both significantly smaller than the predicted value of 6.3 mm.

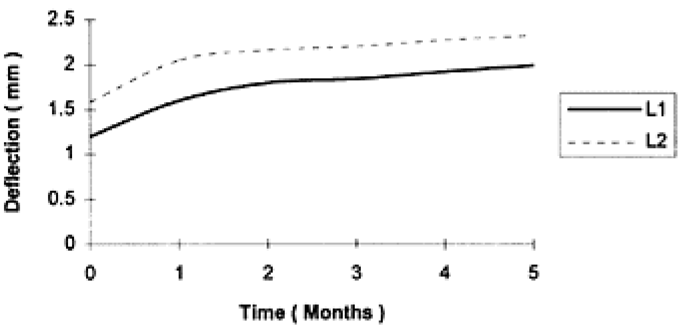

Figure 8 Deflections in Beams L1 & L2 as a function of time

CONCLUSIONS

This paper has described a novel method called Supercover Concrete for preventing reinforcement corrosion in concrete structures using GFRP rebars. The method involves using conventional steel reinforcement together with concrete covers in excess of 100mm, with a limited amount of GFRP rebars in the cover zone. Short term loading tests on single span, simply supported reinforced concrete beams made using this method show:

1. Introducing GFRP rebars in the concrete cover does not significantly affect the structural performance of the member.
2. The GFRP rebars are effective in controlling surface cracking and can be used to reduce surface crack widths to values currently recommended in BS8110 and Eurocode 2.
3. Deflections under working loads are not significantly affected as a result of introducing the GFRP rebars in the cover zone.
4. Existing procedures given in BS8110 can be used to assess structural strength, design crack width and deflections.

It is too early at this stage to draw any conclusions regarding the long term performance of this design method. However, it should be remembered that any deterioration of the GFRP will not compromise the strength of the structure.

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CRACKING OF CARBON FIBRE REINFORCED MORTARS

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ABSTRACT. Portland cement mortar specimens reinforced with pitch based carbon fibres were subjected to bending. In the mixture compositions the volume of fibres varied from 0 up to 3% and 10% of silica fume (SF) was added to a half of the specimens. Load-deflection curves and acoustic emission (AE) counts were recorded. Significant enhancement of the flexural toughness in specimens with SF and sufficient fibre volume was explained by multiple cracking. Values of the relative flexural toughness index corroborated AE records. As conclusions, the applicability of pitch carbon fibres to obtain High Performance Carbon Fibre Reinforced Cement Composites (HPCFRCC) and the possibility of using the AE records for designing HPCFRCC is proposed.

Key words: Cement based composites, Pitch carbon fibres, Silica fume, Acoustic emission, Flexural toughness, Relative toughness Index I_t.

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INTRODUCTION

Carbon fibres exhibit high efficiency as micro-reinforcement due to their strength and dispersion in cement based matrices. As reinforcement they can lead to high flexural and tensile strength, impact resistance and toughness of composite materials. Taking into account also their durablity in various environmental conditions and relatively low cost, in many structures they may be considered as optimum reinforcement, e.g. cladding panels in high rise Ark-Mori building in Tokyo and in Al Shaheed Monument in Damascus, roofing sheets, repairs of concrete structures, foundations for machinery, bridge beams and decks, permanent forms, lightning arrestors, transmission poles, wave absorbers, etc.

One of the first test results on pitch carbon fibres used as dispersed reinforcement for brittle cement based matrices was published in the 1980's. OHAMA et al. [1] tested specimens with various kinds of the fibres and obtained excellent mechanical properties for these composites. According to AKIHAMA et al. [2] the improvement was related to two phenomena: pull-out and fracture of the fibres. NISHIOKA et al. [3] also observed these two different processes and suggested that fibres below 600 MPa tensile strength should not be used as reinforcement. Mechanisms of control of microcracks by carbon fibres were considered by BRANDT and GLINICKI [4], BAYASI [5]. Later, KUCHARSKA [6], KUCHARSKA and BRANDT [7] have shown that with appropriate mixture proportions and sufficient fibre content the development of multiple cracking and the enhancement of the mechanical properties was possible.

Several other problems were also considered:

– uniform dispersion of the fibres and reduction of fibre volume to decrease the cost, QUIJUN ZHENG and CHUNG [8], PU-WOEI CHEN and CHUNG [9],

– influence of the length of fibres on their efficiency, OHAMA et al. [1], PARK et al. [10],

– influence of the kind and content of aggregate, SOROUSHIAN et al. [11].

In this paper, tests on cement mortars reinforced with short pitch-based carbon fibres with and without silica fume reported previously by KUCHARSKA and BRANDT [6], [7], [12], are considered again. Cracking in tested specimens and load-deflection curves related to acoustic emission activity are analysed in detail. Enhancement of flexural toughness of this kind of composites by the phenomenon of multiple cracks as shown in KUCHARSKA [6] is confirmed.

EXPERIMENTAL

The components of the mixes and their proportions have been described in detail in [6] and [7]. The basic mix components were:

– Ordinary Portland cement (OPC) of Polish origin from Rejowiec,

– superplasticizer (SP) with melamine formaldehyde sulfonate as an active ingredient, 1.1% per mass of cement mass,

– low modulus pitch-based carbon fibres (CF) C-1035 produced by Kureha Carbon Industry Co. Ltd., length 3 mm, dia.14.5 μm, f=720 MPa,

– silica fume (SF) from Łaziska in Poland, and

– fine aggregate selected from standard sand with grains<1mm.

Constant ratio of water to cement and silica fume equal to 0.31 was maintained and other data are given in Table 1.

Table 1. Composition of tested mixes

Mix #:	CF0	CF1	CF2	CF3	CF4	CF5	CF0S	SF1S	CF2S	CF3	CF4S	CF5S
Sand/Cement ratio (S/C)	1.50	1.49	1.48	1.47	1.44	1.41	1.37	1.36	1.35	1.34	1.32	1.30
SF [%] per mass of cement	0	0	0	0	0	0	10	10	10	10	10	10
Carbon Fibres V_f [%] vol.	0	0.26	0.52	0.79	1.5	3.0	0	0.26	0.52	0.79	1.5	3.0

The fibres were dispersed using the wet method, [9], and the mixing time was extended up to 15 minutes. An ordinary mortar mixer was used. The plate specimens, cast in flat moulds 140x160 mm and 15 mm deep, were demoulded after 24 hours and were cured in saturated lime water until two days before testing.

Test were carried out after 28 and 56 days. Specimens 15x20x160 mm were sawn out of the plates and subjected to bending. Third-point loading was applied using an Instron 1251 machine with a closed-loop servo control system enabling stable tests to be performed. The tests were carried out under deflection control at rate of 0.05 mm/min until the applied load reached zero or the displacement reached 1 mm, whichever was the first.

The displacement transducer (LVDT) was attached to the specimen at the supports by a special yoke to measure load-point deflections with respect to support points. This method eliminates errors related to rigid movements of the specimen and to other possible parasitic displacements. Load-deflection curves served for calculation of total absorbed energy W_{tot} as the area under the curve up to the end of the test.

Acoustic emissions were recorded using a transducer with a nominal resonance frequency of 200 kHz. The receiving head was situated on the specimen between a support and loading points. In the reported tests the AE equipment recorded all signals above certain level of discrimination, however the magnitude and amplitude of an individual AE event could not be measured. Therefore, after these results it was impossible to execute a deep analysis and to detect the source of the event: crack in the matrix or interface, or fracture of a fibre. Only the number of single events, the rate of increase and the total number at different stages have been recorded. For further analysis two stages of loading were selected: peak load and maximum deflection at the end of the test.

A PC based acquisition system was used within the ASYST environment and sampling rate of 1 Hz was applied.

RESULTS AND DISCUSSION

In Figs. 1 and 2 load-deflection curves and the magnitude of the AE are shown for selected characteristic specimens with different V_f and with or without silica fume (SF). The number of AE events at two stages as mean values of series of 4–6 specimens are given in Table 2.

Table 2. Numbers of AE [100] events in tested specimens

			contents of carbon fibres V_f				
age	levels or load	0	0.26	0.52	0.79	1.5	3.0
28	up to max. load	24	15	25	60		–
days	up to end of test (without SF)	53	1800	2080	2270	–	–
	up to max. load	15	1	38	570		–
	up to end of test (with SF)	108	1650	2300	2300	–	–
56	up to max. load	14	13	9	49	210	200
days	up to end of test (without SF)	370	1466	1300	1910	2000	3250
	up to max. load	29	59	27	460	950	1050
	up to end of test (with SF)	350	1450	2350	2400	3050	3750

In Fig. 3 the influence of fibre content on total AE events (Table 2) and on W_{tot} for specimens with and without SF are shown.

From these results it was found that in all specimens tested very few AE events were detected at low levels of loading before the appearing of the 1st crack. This observation showed that the AE recording was correctly executed and that the lower discrimination level for AE signals had been appropriately selected.

Only single cracks occurred before effective initiation of the damage process, as has been observed by BALAGURU and SHAH [13]. Then, after the change of slope of the load-deflection curve the higher rate of cracking was recorded by AE.

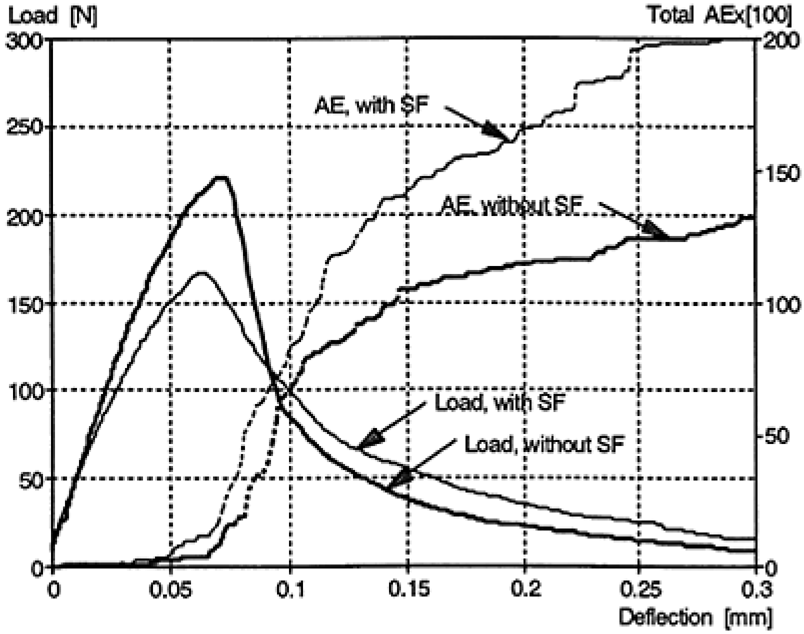

Figure 1 Curves load-deflection and AE-deflection in specimens made of plain matrix (V_f=0%) with and without SF

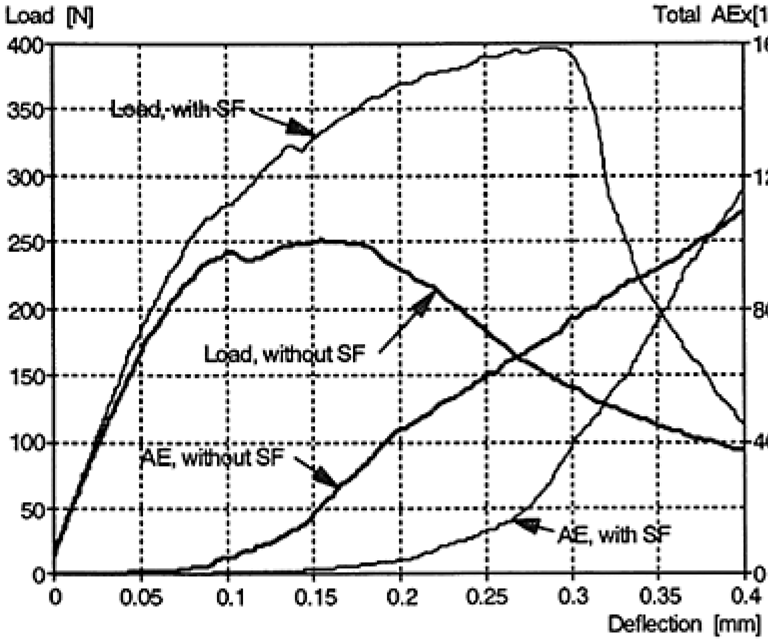

Figure 2 Curves as in Fig.1 but with carbon fibres V_f=1.5%

As can be seen from Figs. 1 and 2 the influence of fibre reinforcement was considerable. In specimens with more fibres higher numbers of AE events occurred and this was due to more intensive but dispersed cracking, Table 2 and Fig. 3. Moreover, the shapes of the AE-deflection curves are different for plain and reinforced specimens. The variations of slope of load-deflection curves which are due to microcracking and later to development of major cracks are correlated with rapid increases in the number of AE events and of absorbed energy W_{tot}, Fig.3.

Systematic analysis of flexural toughness of all specimens tested was carried out using different systems of toughness indices. The most appropriate for estimation of the influence of the volume of fibres and of silica fume on the mechanical properties was the relative toughness index I_t. It is defined as the ratio of the energy absorbed in FRC specimen up to the end of the test to the similar energy in a comparable beam without fibre reinforcement, ACI 544 [14]. This index is free of uncertainty of determination of the 1st crack and, besides, it provides a basic measure of the efficiency of the fibres. Values of I_t for tested specimens (Fig.4) show that the toughness was considerably enhanced for specimens with $V_f \geq 0.79\%$ but only when SF was added.

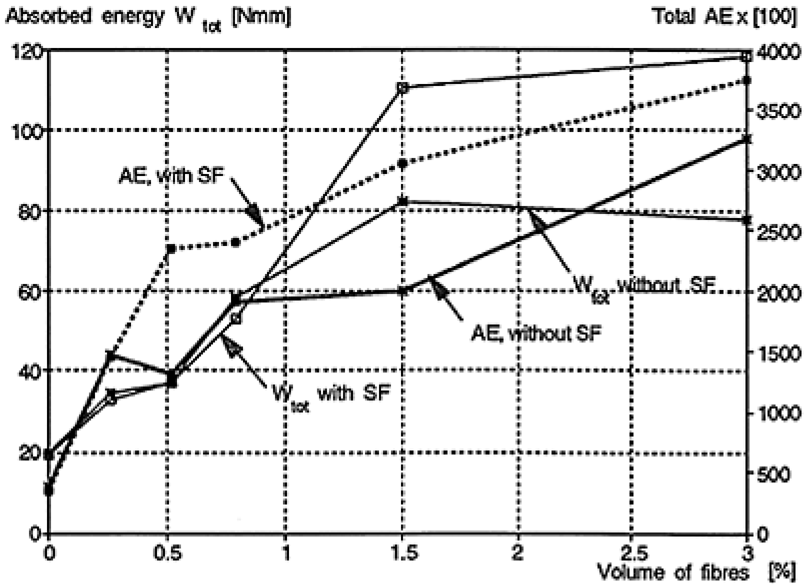

Figure 3 Total absorbed energy W_{tot} and AE counts as functions of volume of fibres

Similarly, significant increase of flexural toughness and of AE events of the specimens with higher volume of fibres and containing SF corroborates the conclusions formulated in [6] that the phenomenon of multiple cracking occurs in high performance mortars with pitch carbon fibres. The influence of SF on recorded cracking process for lower values of V_f was negligible.

The rate of AE events reached a maximum immediately after the peak load, which corroborated observations by MAJI and SHAH [15]. These events were caused by propagation of major cracks. Effective control of crack propagation by the fibres lead to an ability of the composite material to retain a considerable load bearing capacity after peak load, Fig. 5. The curve representing total AE counts exhibits perfect correlation with the load-deflection curve during loading and unloading cycles. Cracks did not propagate in these cycles and specimens exhibited quasi-elastic behaviour. Hysteresis represented by areas of the loops is related to the slipping of the fibres.

From all these diagrams it can be seen that the increase of the fibre content beyond 1.5% does not provide significant improvement of mechanical properties and certain irregularities appear. This is attributed to less uniform dispersion of the fibres, [7].

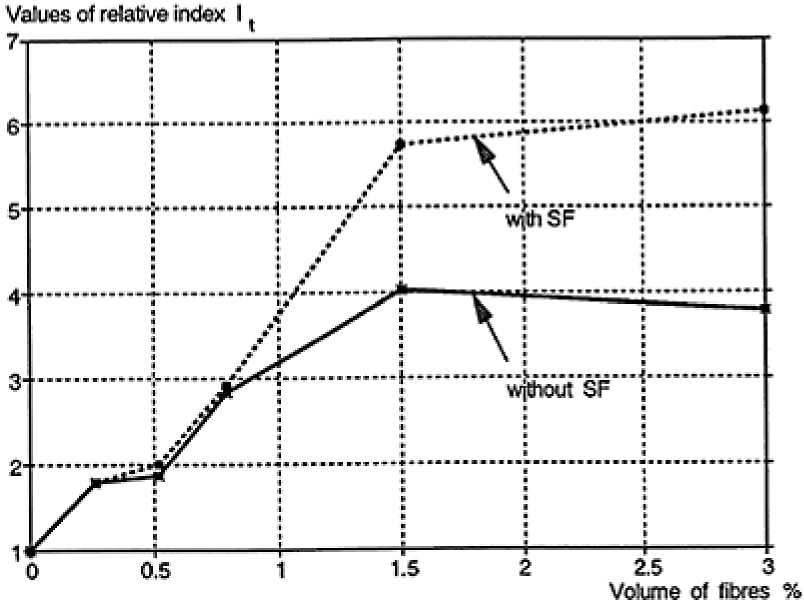

Figure 4 Relative toughness index It as a function of volume of fibres

CONCLUSIONS

Significant enhancement of the flexural toughness of pitch carbon fibre composites may be obtained when multiple cracking ocurs and this requires that the fibre-matrix bond is improved by the addition of a microfiller and that the fibre volume is adequate.

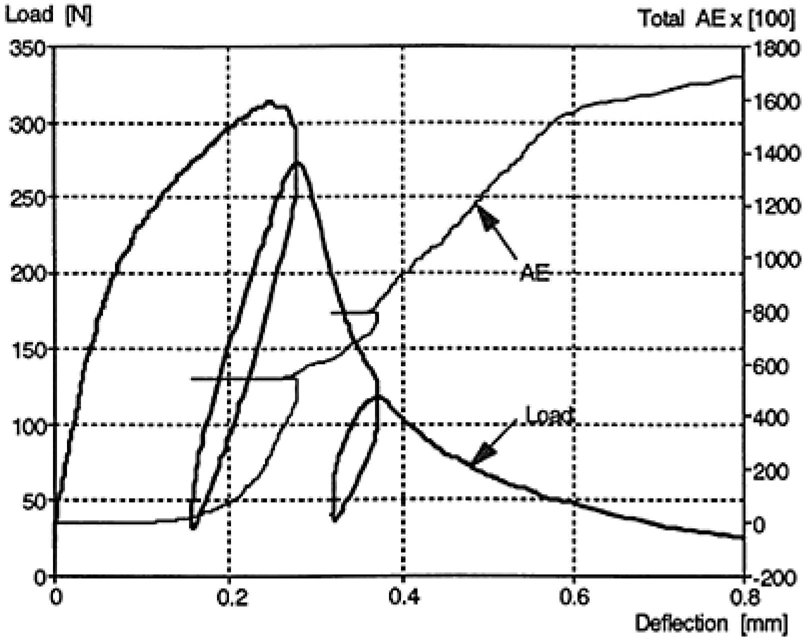

Figure 5 Curves load-deflection and AE-deflection for specimens subjected to unloading and loading cycles in post-peak stage

Determination of appropriate mixture proportions, of conditions of execution and of curing may be purposefully supported by the results of acoustic emission activity records.

Acoustic emission is helpful in following the behaviour of the composite elements during consecutive stages of cracking up to final fracture. Application of more advanced AE equipment would enable the sources of recorded acoustic events to be identified. Such possibilities would extend considerably the effective design of CFRC.

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LOW COST FIBRE REINFORCED CEMENT PRODUCTS BY USING INEXPENSIVE ADDITIVES

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ABSTRACT. The concept of using fibres in mortars dates from the past. Nowadays, fibres from various sources such as asbestos, polypropylene, cellulose, glass and others are succesfully applied to mortars for repair or remaking purposes as well as for a great variety of precast components or thin walls. By this way, plastic and longtime shrinkage defects are avoided resulting in remarkable longevity of materials. The aim of this experimental work is the production of good quality fiber cement products by using cost effective by-products and find alternatives to asbestos cement industry. Flexural, tensile and splitting tensile strength were measured as well as the first crack load. Cracking tendency of composites was estimated by observing with microscope thin plates cured at low humitidy environment 40–45% RH.The parameters which were studied are: a) type and percentage of fibres b) percentage of cement replacement by fly ash c) cementitious content d) addition of sand. Based on results it could be said that there are cost saving alternatives in fiber cement composites. High percentages of cement from 30 to 60 can be replaced by fly ash while the total amount of cementitious can be reduced. Polypropylene seems to render fiber cement grout of good quality.

keywords: Fiber Reinforced cement, Grouts, Fly ash, Natural fibers. Polymeric fibres, Flexural, Tensile, Splitting, Strength.

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INTRODUCTION

Fiber reinforced mortars or grouts are widely applied in cast and precast composites. Especially fibres of low moduli of elasticity such as natural fibres (cellulose, jute, sisal, bamboo, coconut) and polymeric fibres (polypropylene, nylon polyethylene, PVA) have been used since 1960 with high efficiency [1] acting as crack inhibitors. These grout mixes are characterized by high cement content and large volumes of material consumed in situ. Applications of this type are groutings for support lining for mine tunnels or rock slopes or for repairing canals or damaged concrete surfaces as well as for construction of thin section elements or thin (<25mm) plaster coverings.These fibres can also replace secondary steel reinforcement in thin slabs [2] [3]. In precast cement based industry the above mentioned fibres may replace asbestos in pipes, roofing sheets, thin walls or boxes for various uses [2]. Actually they compete asbestos cement products developed in 1900 [3] which according to European Committee directive 91/659 EC L363/91 do not have future since they are considered dangerous to health.

The great advantages attained by fiber cement mortars when randomly distributed fibres added are [2] [3] [4]:

– Increase stress-strain capacity and flexural strength. Under static load—whatever the mode loading- a substantial increase in the area under stressstrain curve is observed.

– Improve toughness and post cracking ductility.

– Act as energy absorbers.

– Reduce plastic shrinkage of first hours after placing and inhibit cracking.

– Improve shear strength. An increase of 17% in shear strength due to polypropylene fibres has been found experimentally [5].

– Exhibit better bond to substrate. Therefore the inherent defect of cementitious matrices, brittleness, is overcome. This is of special importance in case of grouts or slurries where there are no aggregates to hinder crack propagation.

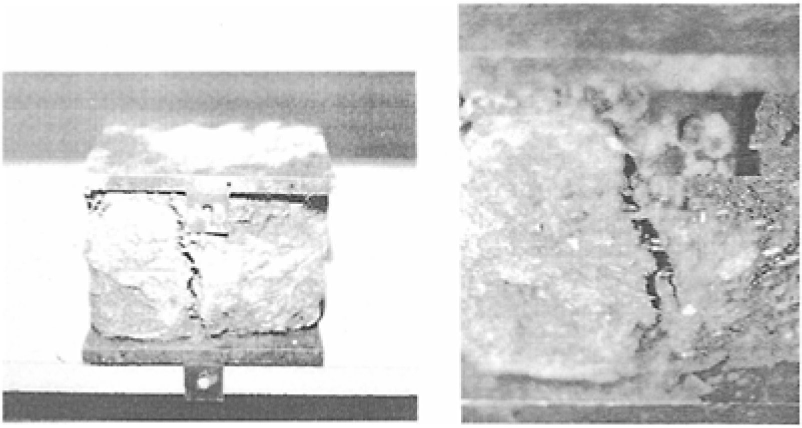

Figure 1. Cruhing model of mud bricks

The concept of reinforcing low potential materials with fibres dates from the past [6]. Fig. 1 shows the characteristic behaviour of chopped straw in old sunbaked mud bricks. The fibres cause slow development of a wide fracture zone. They tranfer stress across cracks by bridging them. Therefore the materials can retain some residual post crack strength and fragmentation is inhibited [2] [3]. The good performance of fibres in a given mortar depends mostly on fibres content, Vf (percent by volume of composite), fiber geometry (aspect ratio: fiber length/fiber diam. and distribution of fibres) [2] [3] [4] [6]. Different techniques such as Hats check method and wet-mix shotcreting in precast and in cast application are the most known present day processes of manufacturing fiber cement composites but much amelioration could be made [2] [3].

SCOPE

The aim of this research programme was firstly to find alternatives to asbestos cement industry ELLENIT, one of the remaining factories of ETERNIT producing asbestos cement products. The objectives were a) to reduce the cost of production by replacing a substantial proportion of cement with low cost fly ash -locally produced- and b) to test the performance of alternatives in conjuction with cement substitution for fly ash. Before proceeding to full scale tests in industry, an experimental work was carried out at Lab. of Building Materials of University of Thessaloniki in order to decide upon proportioning parameters. A number of 29 grout and slurry mixtures were prepared and tested. The result of this part of research work could find direct application in cast fiber reinforced mortar, grout or slurry production.

EXPERIMENTAL DETAILS

Materials

Portland cement 145 type was used in combination with greek fly ash wich is a High—Calcium Fly Ash (HCFA). The raw HCFA has a free lime content of 9–10% but it is ground and treated in the mill with equivalent quantity of H20 so as the free lime content of the final HCFA decreases to 3% (CaO+H20→Ca(OH)2). The suitability of HCFA in slurry and grout mixtures was found in previous experimental work [7]. Chemical composition of the cementitious materials is given in table 1. Limestone filler or river sand of max. size up to 1 mm were used in grouts. In asbestos slurries dry waste was added to limestone filler according to practice followed in asbestos cement industry.

Apart from asbestos (chrysotile) used in two densities $25{\cdot}10^3$ and $45{\cdot}10^3 Kg/m^3$, three other type of fibres were tested: cellulose, fibrilated polypropylene, and PVA. Although the mechanical performance of mortars with these fibers is inferior to steel or glass fiber mortars [3] they were selected because of their better behaviour in grouts and slurries. Cellulose mortars present high water rententivity and standability while polypropylene fibres are very effective in reducing shrinkage and have excellent resistance to moisture, acids and alkalis [2] [3]. Both of them are relatively inexpensive materials. In addition, HCFA

Table 1. Characteristics of cementitious materials used in fiber reinforced grouts

Constituents	Portlanl cement 145 %	Ptolemaida fly ash mean value
SiO_2	19.70	32.5
Al_2O_3	4.80	11.3
Fe_2O_3	3.70	5.02
CaO	54.10	$30.6\ \frac{avai/9.0}{free\,3.0}$
MgO	1.70	2.3
SO_3	2.80	6.5
Na_2O	0.15	0.13
K_2O	0.60	0.52
TiO_2	–	0.65
Loss of ignition	5.4	4.7
Specif. Gravity	3.1	2.47
Percent retained on 4900 mesh sieve	0.2	15–20

added to cement mixtures for cement replacement reduces the pH of mixture with time. This is advatageous for cellulose which is sensitive to high alkaline environment of hydrated cement.

The characteristics of the used fibres, have been provided by the suppliers.

Polypropylene
Type: PPC3H6 fibrilated
Length: 12mm
Diam.: $18 \cdot 10^3$ mm
Spec, grav.: $0.9 \cdot 10^3$ Kg/m^3
Harmless
Tens, streng.: $3–4x10^2$MPa
Mod. of elastic.: $6–9\times10^3$MPa

Natural cellulose
Type: ARBOCEL PZ8
Length: 1, 4 mm
Moist.cont.: 10–12%
Spec, grav.: $1.5 \cdot 10^3$ Kg/m^3
Harmless
Resistant to diluted acids and alkalis
Tens, streng. $3-5\times10^2$ MPa
Mod. of elast. $8–10\times10^3$ MPa

Asbestos
Type: chrysotile (ZIDASBEST S.A.)
Diam: $0.05–5 \cdot 10^3$ mm
Spec.gr.: $2.5–10^3$ Kg/m^3
Tens, strength: $164 \cdot 10^3$MPa
Dangerous in health when inhaled

PVA
Type: V-10 (UNITIKA NEWLON A.A)
Length: 6mm
Diam.: 1.6d
Harmless

Proportioning

The mix proportions of all mixtures are shown in Table 2. Instructions for mixing the constituents were given by the supplier of fibres. Usually, fibres were added in form of slurries. Cement was used in different quantities. Water amount was adjusted for achieving adequate workability of mortars or fluitidy of grouts and slurries.

Two triplets of 40×40×160 mm prisms were tested for flexural strength of each age. A set of four briquettes (known from cement tensile tests) was used for direct tensile strength determination. Two cylinders 50×100 mm were used for splitting tensile strength loaded according to ASTM C-496. The parts of prisms after testing in flexure were pressed in order to measure the first crack load. Dynamic modulus of elasticity was estimated by measuring ultrasonic waves velocity through Ferret prisms by sonometer. The compaction of fresh mixtures was made at the vibrating table described in cement regulations for preparing cement mortars. With mixtures of high content in fibres and especially with cellulose, stiffness problems appeared. A superplasticizer of lignosulfonate origin was used up to 1% by weight of cementitious to anticipate them. Specimens were cured in a climate chamber at 45% Relat. Humidity and 25C°. This realistic environment was chosen to be similar to curing conditions of precast products and construction conditions in summer period. In addition, four plates 400×400 mm of 6mm thickness made with the four types of fibres (Mixes No 9, 27, 28, 29), were casted in order to see the appearance and resistance to cracking. The plates were placed at the above mentioned environment and they were examined microscopically (x100 magnification) at regular periods for over 3 months to see cracks or other deformations.

RESULTS AND DISCUSSION

Mean values of test results are shown in Tables 3, 4, 5 and 6. It can be said that relatively great deviations from the mean values up to 19% were observed especially in mixtures of high fibre content and tensile strength determinations. Asbestos fiber reinforced composites have a specific gravity around $0.55 \cdot 10^3$ Kg/m^3 at a total quantity of cementitious 75% w/w in dry mix. Higher strengths in tension exhibit the slurries with the lower water/cementitious (w/cmtt) ratio, 1.93 (fig. 2). Comparing mixes No 8 and 9 it seems that 40% w/w replacement of cement by HCFA results in a 15% reduction in 28-day flexural strength but at significant increase at 3-months strength. At high w/cement ratios of 2.47 or 5.80, HCFA addition up to 50% w/w of cement does not differentiate mechanical characteristics. Net HCFA slurries developed at 28 days flexural strength around 70% of corresponding of net cement slurries of the same w/cement ratio. Therefore the substitution of cement with HCFA should be made according to the required early strength and final strength of the product.

Cellulose fiber reinforced grouts have a specific gravity around $0.60 \cdot 10^3 Kg/cm^3$ for a cementitious content in dry mix of 70%. In comparison with asbestos mixtures of almost the same w/cement ratio higher strength can be achieved with addition of a proportion 0.34% cellulose in dry mix. HCFA percentages up to 60 contribute effectively to flexural strength development especially at

Table 2. Mix proportions of fiber reinforced cement grouts and slurries

Mix ture Code No	Cement	HCFA	Water/ cement.	Asbestos ehysot.	Cellulose	Polyprop.	P.V.A.	Limest. filler +Dry waste	Sand (max. size 1 mm)	Superplas. (% w/w of cement.)
1	1	–	5.8	0.25	–	–	–	0.08	–	–
2	1	–	6.6	0.25	–	–	–	0.08	–	–
3					–	–	–			
4	–	1	5.8	0.25	–	–	–	0.08	–	–
5	0.6	0.4	5.8	0.25	–	–	–	0.08	–	–
6	0.5	0.5	5.8	0.25	–	–	–	0.08	–	–

7	0.6	0.4	2.47	0.25	–	–	–	0.08	–	–
8	1	–	1.93	0.25	–	–	–	0.08	–	–
9	0.6	0.4	1.93	0.25	–	–	–	0.08	–	–
10	1	–	2.71	–	0.34	–	–	0.08	–	–
11	0.6	0.4	2.71	–	0.34	–	–	0.08	–	–
12	0.5	0.5	2.71	–	0.34	–	–	0.08	–	–
13	0.4	0.6	2.71	–	0.34	–	–	0.08	–	–
14	1	–	0.86	–	–	0.072	–	1.0	–	–
15	0.6	0.4	0.86	–	–	0.072	–	1.0	–	–
16	1	–	1.01	–	–	0.04	–	1.08	–	–
17	0.6	0.4	1.09	–	–	0.04	–	1.08	–	–
18	1	–	1.56	–	–	0.07	–	2.78	–	10
19	0.5	0.5	1.56	–	–	0.07	–	–	2.78	10
20	0.4	0.6	1.56	–	–	0.07	–	–	2.78	10
21	1	–	1.56	–	–	0.07	–	–	2.78	10
22	0.6	0.4	1.09	–	–	0.04	–	–	1.08	10
23	1	–	0.86	–	–	0.05	–	1.0	–	10
24	1	–	1.56	–	–	–	0.077	2.78	–	10
25	1	–	1.56	–	–	–	0.077	–	2.78	10
26	0.4	0.6	1.56	–	–	–	0.077	–	2.78	10
27	0.6	0.4	1.56	–	–	–	0.077	2.78	–	10
28	0.6	0.4	1.56	–	–	0.07	–	–	2.78	10
29	0.6	0.4	2.71	–	0.34	–	–	0.08	–	–

Table 3. Mechanical characteristics. Asbestos slurries.

Mixt. Code No	Flexural. Strength MPa 28 days	Tensile Strength MPa 28 days	Split Tenslile Streength MPa 28 days	First crack load in compression MPa 28 days	Specif. Grav. 10^3 kg/cm^3 28 days	E dyn MPa 28 days
1	0.64	–	–	0.34	0.69	3.40
2	0.47	–	0.13	0.57	0.55	2.80
4	0.45	–	0.23	0.16	0.45	3.96
5	0.41	–	–	0.16	0.33	–
6	0.43	–	0.05	0.17	0.30	3.90
7	0.85	0.45	0.34	0.85	0.56	2.40
8	2.00 (2.8)	0.71 (1.08)	0.40 (0.90)	3.00 (3.80)	0.66	2.32
9	1.70 (3.01)	0.58 (1.10)	0.49 (0.85)	2.70 (4.10)	0.61	1.52

-*Values in parenthesis correspond to 3-months strength

Table 4. Mechanical characteristics. Cellulose grouts.

Mixt. Code No	Flexural Strength MPa		Tensile Strength MPa	Splitting Tens. Str. MPa	First crack load in compression MPa		Specif. Gravity 10^3 kg/cm^3		E dyn. GPa	
	28 days	2 months	28 days	28 days	28 days	2 months	28 days	2 months	28 days	2 months
10	1.03	1.95	0.45	0.37	1.65	2.05	0.613	0.570	2.80	8.20
11	1.31	2.02	0.57	0.40	1.60	2.00	0.610	0.580	4.67	12.10
12	1.60	2.12	0.43	0.34	1.61	2.00	0.608	0.567	7.00	6.90
13	1.31	1.66	0.31	0.30	1.63	2.30	0.597	0.590	6.83	7.20
29	1.58	–	0.35	–	1.90	–	0.600	–	7.70	–

Table 5. Mechanichal characteristics. Polypropylene grouts.

Mixt. Code No	Flexural Strength MPa			Tensile Str. MPa	Split. Str. MPa	First crack load in compression MPa			Specific Gravity 10^3 Kg/cm^3			E dynamic		
	7 days	28 days	3 months	28 days	28 days	7 days	28 days	3 months	7 days	28 days	3 months	7 days	28 days	3 months
14	11.00	7.90	11.60	–	–	10.20	10.80	10.00	1.27	1.30	1.29	4.65	7.12	7.08
15	9.10	6.60	11.20	–	–	7.3	13.40	8.70	1.29	1.26	1.32	4.69	6.30	7.30
16	3.40	5.60	8.30	1.90	1.70	5.22	9.30	9.60	1.19	1.20	1.19	4.70	5.37	5.94
17	2.28	4.10	5.30	0.85	1.50	3.50	5.50	7.00	1.12	1.19	1.81	3.10	3.60	3.80
18	2.10	2.70	3.30	0.80	1.14	2.90	3.80	4.50	1.28	1.29	1.35	4.10	3.45	5.50
19	2.00	3.20	3.80	0.43	1.27	2.70	4.90	6.10	1.26	1.31	1.41	2.77	3.10	4.97
20	2.30	2.40	2.40	0.35	0.80	2.90	4.60	5.60	1.41	1.32	1.37	3.06	2.98	4.51
21	3.20	4.00	4.80	0.84	1.97	3.20	3.50	3.20	1.11	1.50	1.20	2.50	5.20	3.57
22	3.40	5.50	5.80	1.20	1.80	4.70	8.70	8.50	1.24	1.50	1.42	3.40	–	7.80
23	6.50	6.50	9.10	–	–	8.72	17.70	10.70	1.20	1.50	1.40	5.20	–	5.08
28	3.80	4.90	–	1.07	1.85	3.40	4.05	–	1.48	1.27	–	3.82	4.23	–

Table 6. Mechanical characteristics. P.V.A. grouts.

Mixt Code No	Flexural Strength (MPa)			Tensile Str. (MPa)	Split. Str. (MPa)	First crack load in compression (MPa)			Specific Gravity 10^3 Kg/cm^3			E dynamic		
	7	28	3	28	28	7	28	3	7	28	3	7	28	3

	days	days	months	days	days	days	days	months	days	days	months	days	days	months
24	8.95	9.38	7.20	1.22	1.90	7.30	9.50	7.80	1.33	1.41	1.39	4.65	6.70	6.12
25	2.30	3.50	3.18	1.25	1.13	4.40	6.10	6.10	1.41	1.50	1.50	4.80	8.50	8.56
26	1.00	2.00	1.50	0.21	0.61	2.20	3.20	2.70	1.33	1.40	1.41	1.85	1.82	3.67
27	3.60	5.70	–	1.10	–	3.80	6.70	–	1.60	–	–	4.33	4.33	–

the age of 2 months. Flexural strength and first crack load seem to be more influenced by cellulose fibres.

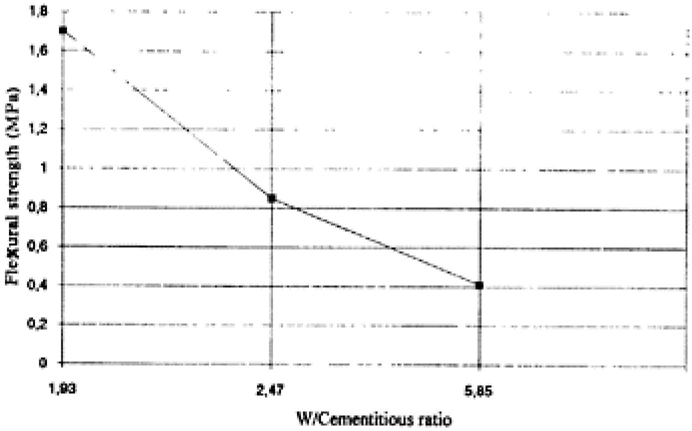

Figure 2 Asbestos cement slurries. Effect of w/cementitious on flexural streng

Polypropylene fiber reinforced grouts with 48% w/w cementitious content in dry mix, exhibited the higher strength values of 10 MPa with a proportion of fiber content 0.072 and w/cement ratio from 0.86 to 1.09 (mixes No 14, 15, 16). Even with lower proportion of fibres 0.040 the values for strength in tension and compression remained higher in comparison with those of asbestos and cellulose mixtures, although cementitious content in dry mix was significantly reduced (26% w/w of dry mix). Fig.3 shows the effect of fiber content in flexural strength. Addition of HCFA up to 50% does not change

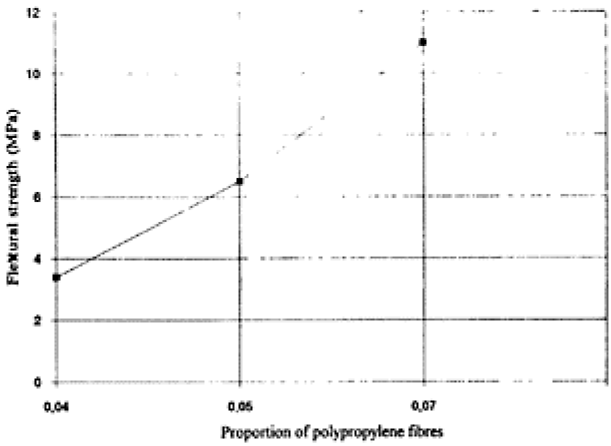

Figure 3 Polypropylene cement grouts. Effects of proportions of fibres on flexural strength

considerably even the 7-day strength and it seems that HCFA cooperates very well with polypropylene fibres in early and mainly longterm strength development. Comparing pairs of mixes No 18, 21 and 17, 22 it is obvious that limestone filler could be advantageously replaced by sand. Addition of superplasticizer 1% by weight of cementitious content has a significantly positive effect in strength development by improving the fluidity of grouts, (mixes No 17, 22). The specific gravity of polypropylene fiber reinforced composites ranges from $1.25 \cdot 10^3$ Kg/m^3 to $1.35 \cdot 10^3$ Kg/m^3 according to cementitious content.

All PVA fiber reinforced composites showed a drop in strength (or no increase) at the age of 3 months. This implies negative effects in the system cement/PVA fibres and further research is needed. HCFA addition in cementitious content is not advatageous as in the case of previous mentioned fibres. An obvious reduction in strength is observed (mixes No 25, 26 and 24, 27). Limestone filler renders higher strength grouts in comparison with corresponding mixtures with the same content of sand.

None of the plates examined by microscope showed cracks up to the age of 3 months.

CONCLUSIONS

1. In asbestos fiber reinforced slurries, cement could be replaced up to 30–40 percent by HCFA. A reduction of 15–20% in strength should be taken into account at early ages (7, 28 days). HCFA cooperates very well with cellulose fibres and could replace cement up to 60% by weight. High quality fiber reinforced composites (up to 10MPa flexural strength) can be produced by using polypropylene fibres. HCFA could be added up to 60% by weight of cement in polypropylene grouts. Mixture of low cementitious content in dry mix (26%) developed high flexural strength up to 3.0

MPa. Sand could be used as an alternative for limestone filler. All these lead to the conclusion that cost effective and high quality grouts or composites could be produced by using polypropylene fibres.

2. HCFA seems to cooperate very well with all examined fibres except of PVA fibres.
3. Superplasticisers are very effective in fiber grout mixtures since they improve fluidity and enhance strength development.
4. Based on data given by asbestos cement industry ELLENIT and a short time market research the following economical considerations could be made: The replacement of cement by HCFA up to 40% results in the 17% cost saving independingly the type fibrous materials used.

    The alternative of using polypropylene fibres instead of asbestos rises the cost of slurry production up to 200% approximately but the resulting final cement products are of high quality in comparison with the technical characteristics of asbestos cement products.

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BIAXIAL FRACTURE BEHAVIOUR OF FIBRE REINFORCED CONCRETE

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ABSTRACT: The fracture properties of Polypropylene-, glass- and steel-fiber rein-forced concrete (FRC) have been determined with a new wedge splitting procedure us-ing cubic specimens with a deep notch under uniaxial and biaxial (tension-compression) loading. Under uniaxial loading conditions, the notch bending tensile strength of plain concrete and FRC is similar. For biaxial loading, however, the measured values of FRC are approximately 10% lower than those of plain concrete. The crack growth resistance, which is characterized by the specific fracture energy, is 1.5 to 10 times higher for the different FRC's under uniaxial loading than for plain concrete. For biaxial loading, the increase is by a factor of 1.2 and 4.5. The changing fracture properties at uniaxial and biaxial loading conditions are described and explained by modeling assumptions. Some important directions to assess the fracture properties of fiber reinforced concrete are mentioned for the practical engineer.

Keywords: Polypropylene-, glass-, steel- fiber reinforced concrete, uniaxial and biax-ial fracture behaviour, specific fracture energy, uniaxial and biaxial splitting test

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INTRODUCTION

Though the compression strength of concrete is high its tensile strength is rather low and responsible for crack initiation and damage of concrete structures in many cases. Therefore it is tried to compensate this draw-back by reinforcing metallic or nonmetallic fibers (fiber reinforced concrete, abbreviated FRC in the following). The main aim of adding fibers is to improve especially the tensile strength, the impact strength and the fracture toughness. Many investigations have been performed in order to study the possibilities of making use of different kinds of fiber materials and fiber types. In this context, the influence of volume fraction, length and cross section of the fibers on the mechanical properties have been measured and discussed in the literature. Relatively few investigations exist on the fracture behaviour in comparison with the other mechanical properties like compression strength, bending strength etc. Most works (for example [1–5]) treat the uniaxial fracture behaviour of FRC, some exist on two-axial and multi-axial compressive loading [6–11] and very few on the fracture properties of FRC during twoaxial tension-compression loading, though this is the type of loading, which leads to crack initiation and propagation in most cases. Therefore failure envelopes have been determined in tension-compression loading experiments of unnotched specimens made of FRC in [12, 13]. The influence of fiber length and number of fibers on the fracture properties of Polypropylene FRC [14] and steel-fiber FRC [15] has been measured with notched specimens. In addition, the biaxial fracture behaviour has been simulated theo-retically in [16]. The main result of these studies is that, the notch bending strength is only slightly influenced by a superimposed compression load during combined loading conditions, whereas the specific fracture energy and especially the post-peak behaviour of the load-displacement curves is influenced much.

In the present study the influence of a combined bending tension and a compression load on the fracture behaviour of identical medium-strength concrete with and without glass fiber or Polypropylene or steel fiber reinforcement has been investigated. The used specimen shape and the type of loading are shown schematically in Fig. 1. The com-pression stress σ_1 is kept constant during the fracture test. A new wedge splitting procedure according to Tschegg [17, 18] has been used which allows uniaxial and biaxial tests with cubic specimens in an easy and convenient way.

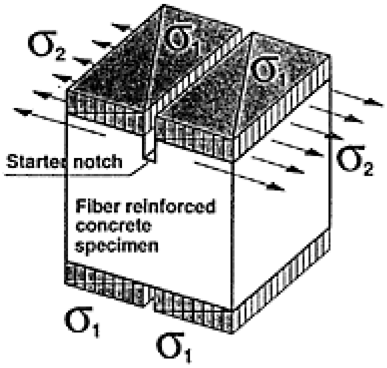

Figure 1: Biaxial loading state of the specimen during the fracture test

TESTING METHOD AND EXPERIMENTAL PROCEDURE

The basis setup for biaxial loading is the wedge splitting method according to [17, 19, 20] for conventional uniaxial mode I fracture testing as shown in Fig. 2, part A. A slen-der wedge, which is positioned in a groove on top of the specimen, splits this specimen. The crack starts from a starter notch and ends on the opposite side along a linear support piece. Friction losses during load transfer are reduced by using needle bearings, so that the measuring results are hardly influenced. Load displacement and crack mouth open ing displacement (CMOD) are measured on both ends of the groove with two displacement gages (LVDT's). The LVDT's are fixed on a frame, which is connected with the specimen only. More details are reported in [20].

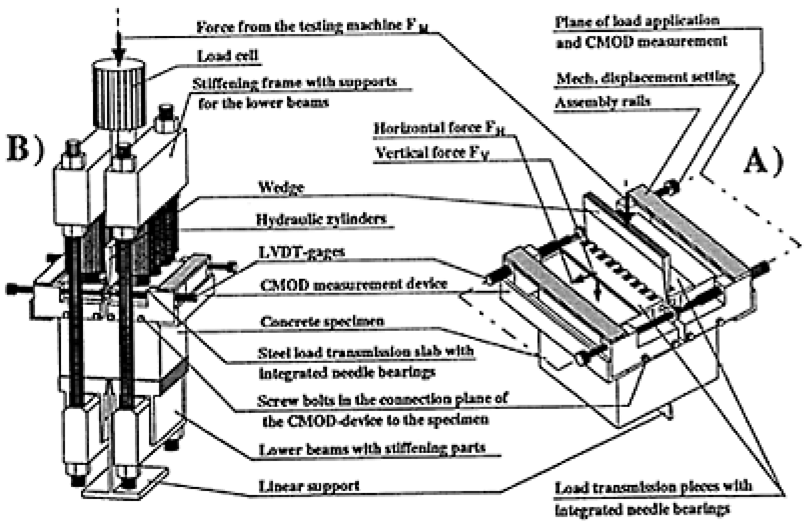

Figure 2: View of the uniaxial splitting method (part A) and the biaxial loading equipment (part B)

The biaxial loading system [21, 22], which is shown in Fig.2, part consists of an uniaxial basis setup (part A) and an additional hydraulic compression load unit in order to generate the biaxial stress state in the specimen. The compression load unit consists of two mechanically independent frames. Three hydraulic cylinders are fixed to both of them. A constant and homogeneous compressive load σ_1 is generated in the specimen in this way [22].

Following construction and functional elements of the uniaxial and biaxial load unit are the same:

- Stiff properties of the whole load transfer equipment owing to the wedge load system
- slender wedge
- load transfer via a needle bearing with negligible friction losses
- load transfer from testing machine to loading system of the specimen
- ligament height and area as well as specimen shape, besides upper part of specimen (see Fig. 3)

Different are:

- The load transmission pieces are L—shaped for the uniaxial test. For the biaxial test, they are part of the load transmission slabs in order to generate the stress σ_1 in the specimen.
- The linear support is a thin and low bar in the uniaxial test, whereas a long sword-like piece is introduced between the two lower beams (Fig. part B) in the biaxial test, without touching them or inhibiting the splitting procedure. Thus the specimen is supported essentially similar as in the uniaxial test.

An important criterion to ensure a homogeneous and well defined biaxial state of stress in the specimen, is to balance the transverse strain between concrete and steel. This problem in treated in [22] in a fundamental way. An appropriate and optimum equipment to balance the transverse strain is obtained by introducing a Teflon foil (approximately 0.5 mm thick) and a cardboard (approximately 1–2 mm thick) between the polished steel load transmission slabs and the specimen. This constructive solution is simple, inexpensive and accomplishes all requirements. More details of the new biaxial loading system are described extensively and discussed in [21, 22].

DATA ACQUISITION AND EVALUATION

In order to fully characterize the fracture behaviour of FRC, load-displacement curves (horizontal, i.e. splitting force vs. load-displacement or CMOD) are recorded until complete specimen separation during stable crack propagation in the uniaxial and biaxial fracture testing equipment takes place. Such curves contain all informations, which are necessary to characterize the fracture behaviour of FRC completely.

The horizontal, i.e. splitting force is determined in the following way: The force FM coming from the testing machine, is measured with a load cell (see Fig. 2). From this the horizontal, i.e. splitting force FH is calculated according to $F_H=F_M/(2tg\ \alpha)$, with $\alpha=5°$ being the wedge angle.

The load-displacements CMOD1 and CMOD2 are measured on both ends of the starter notch and from these the arithmetic mean value is determined. If the two values differ by more than 20 %, the crack front has not grown parallel to the starter notch and the linear support, respectively. These measurements therefore are not considered any more for the evaluation. Splitting force and CMOD values are stored by a modern data acquisition system. These data represent the load-displacement curves, from which the following characteristic fracture data are obtained: (i) the specific fracture energy G_f and (ii) the nominal notch bending tensile strength σ_{2NBS}. The specific fracture energy G_f repre-sents the fracture energy, which is necessary to separate the specimen completely (this energy is proportional to the area under the load-displacement curve) divided by the fracture area (normal projection of the fracture area or ligament area, respectively). The specific fracture energy characterizes the resistance against crack prop agation. The notch bending tensile strength σ_{2NBS} is calculated linear-elastically, assuming that the influ-ence of the vertical force component F_v and the specimen weight are negligible.

MATERIAL, SPECIMEN PREPARATION AND TESTING CONDITIONS

Main aim of the present study is to quantify the influence of the fiber reinforcement on the fracture properties. Concrete composition and storing conditions therefore were kept constant for all test series. The fibers only were varied as to their material, length and additions. Composition of the four test materials as well as their strength properties after 28 days water storage are summarized in Table 1.

Quality and shape of fibers: The diameter of the Polypropylene fibers “HAREX-Poly Con” is 10–20 µm. The glass fibers being called “Cern-Fil-AR-Glass fiber” have the same diameter. The steel fibers are called “HAREX-Steel fiber”. Their diameter is 0.8 mm and they are not anchored at the ends. The authors are aware of the fact that glass fiber reinforcement usually is not used for concrete but rather for mortar and in much higher concentrations there than in this study. The results of this study therefore are of academic value only as to the glass fiber reinforcement. The results, obtained with steel and Polypropylene fibers on the contrary, are of practical relevance too.

Table 1: Properties of concrete mixes after 28 days water storage

Type of aggregates: Natural gravel, limestone Maximum aggregate size: 16 mm		
Cement content	240	kg/m^3
Water content	174	kg/m^3
Water/cement ratio		0.725
Density	2430	kg/m^3
Young’s modulus	~30	GPa

Type	Fiber content Vol %	Fiber length mm	Compr.strength N/mm^2	Flexural strength N/mm^2
Plain Concrete	–	–	24.8	5.6
Polypropylene.	0.5	20	26.2	4.9
Glass FRC	0.5	25	22.2	5.2
Steel-FRC	0.5	32	27.5	5.8

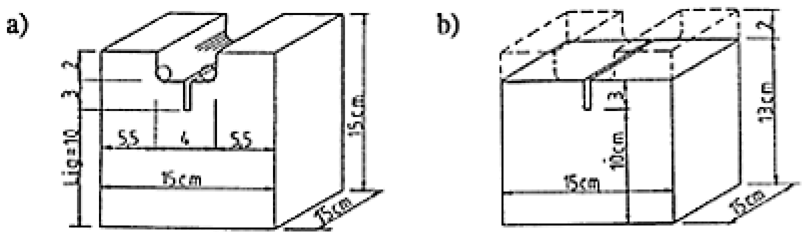

Figure 3: Specimen shape and dimensions a) uniaxial test and b) biaxial test

Specimen shape and dimensions are shown in Fig. 3. Ligament and all other dimensions as well as place and height of load application lines do not differ for uniaxial and biaxial specimens. The top of the specimen, however, is different, as the load is applied via slabs in the biaxial system, which are not needed for uniaxial loading. The specimens were cast to boards (70 cmx15 cmx15 cm), removed from the molds after 24–36 hours and stored in water for 28 days. Before testing the specimens were cut to their final shape (see Fig. 3) and a notch being 2.5 to 3 mm wide was introduced.

Testing was performed in a mechanically driven machine (SCHENCK) with a load capacity of 100 kN. The cross-head speed was 0.5 mm/min in all cases, which approximately corresponds to the RILEM [27] recommendation for fracture tests. The testing temperature was 20–22 °C. To obtain appropriate statistic values, testing was performed with 3–4 specimens for each type of FRC and for each loading condition.

RESULTS AND DISCUSSION

Load-displacement curves for uniaxial and biaxial testing of Polypropylene FRC are shown in Fig. 4. Compared to plain concrete, FRC is characterized by a pronounced softening behaviour in the post-peak regime for uniaxial loading (σ_1/f_c=0%, with f_c being the compressive strength). The softening behaviour is less pronounced for biaxial loading (σ_1/f_c>10%) and decreases with increasing biaxiality, though this reduction is less pronounced. The peak heights of the load-displacement curves for the different biaxial loading conditions are not too different. A similar behaviour has been observed in this study for glass FRC and steel FRC, so that these load-displacement curves are not shown separately.

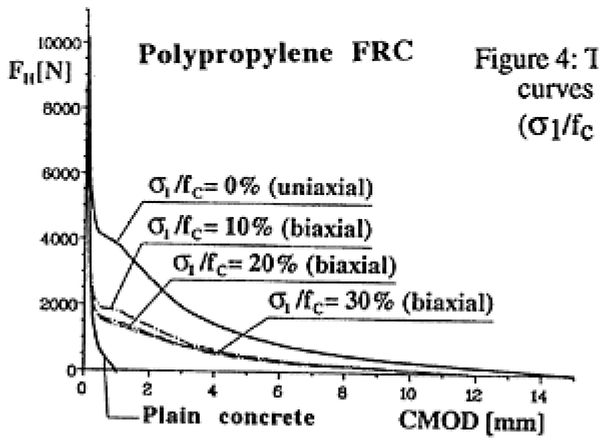

Figure 4: Typical load-displacement of Polypropylene FRC (σ_1/f_c=compr.stress/compr.strength)

The characteristic fracture values Gf and σ_{2NBS} are plotted versus the normalized compressive stress σ_1/f_c with and without fiber reinforcements as parameter, in Figs. 5 and 6. Fig. 5 shows that the uniaxial notch-bending tensile strength σ_{2NBS} is not changed by the fiber reinforcement. For biaxial loading, the values of the FRC are approximately 10% lower than of plain concrete. The curves are almost horizontal between 10 and 60%

normalized compressive stress, which is typical for FRC and plain concrete with varying composition [22–25].

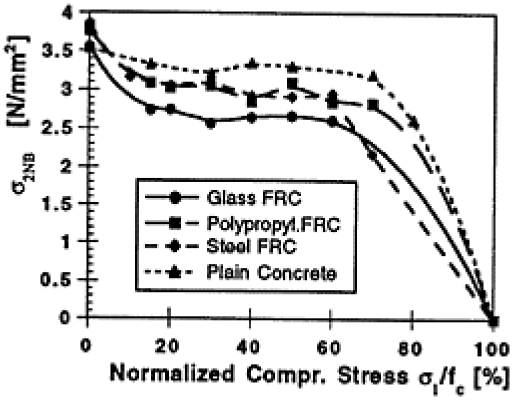

Figure 5: Failure envelope for different FRC's

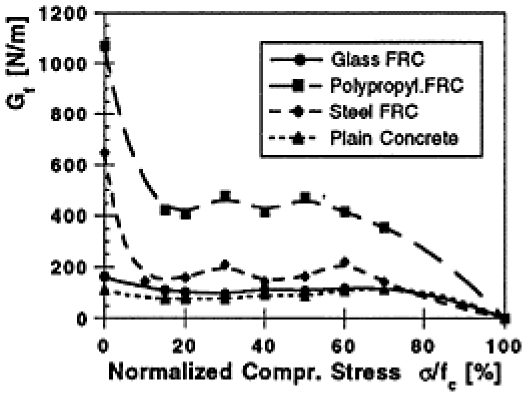

Figure 6: Specific fracture energy G_f for different FRC's

The G_f curves in Fig. 6 demonstrate that the crack growth resistance of FRC's are much higher compared to plain concrete for uniaxial loading (σ_1/f_c=0). The ratios are: $G_{f,}$ poly:$G_{f,steel}$: $G_{f,glass}$:$G_{f,plain}$=9.4:5.7:1.45:1, which means that the G_f value is increased by a factor of 10 by the addition of Polypropylene fibers. These ratios are lower for biaxial loading. For σ_1/f_c being between 20 and 60% the ratios are only 4.5: 1.6:1.2:1, showing that Gf is approximately 5 times higher for Polypropylene FRC. The maximum decrease of the specific fracture energy is found for steel FRC. It is worthy to note that glass fibers increase the fracture properties only slightly. This FRC is not used for practical purposes.

It is of interest for the practical engineer to know, that FRC looses more than 50% of its crack growth resistance under biaxial loading in comparison to uniaxial loading. If G_f values of uniaxial loading tests are used for calculations, extremely unsafe values are obtained, as too high G_f values have been assumed. This result demonstrates that biaxial fracture properties of FRC have to be measured for practical applications, instead of only introducing uniaxial values or substituting them by subtracting some undefined value.

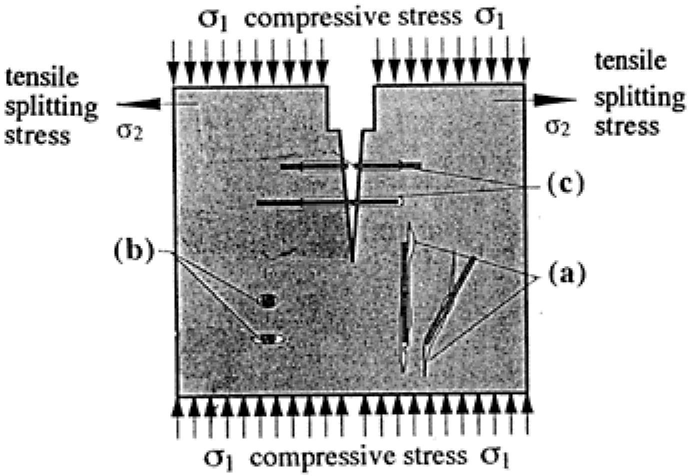

Figure 7: Typical fracture mechanisms for crack propagation under a defined biaxial stress state of fibers in FRC. The fibers are oriented: (a) normal to the notch root (or crack front) and parallel to crack growth direction. (b) parallel to notch root (or crack front) and normal to the crack growth direction (c) normal to notch root (or crack front) and normal to the crack growth direction

The remarkable reduction of the specific fracture energy under biaxial loading may be explained with a simple model according to [14–16]. Though the fibers are randomly oriented in concrete, it is useful to assume three main orientations in order to model the fracture energy consumption. These three orientations are shown schematically in Fig. 7: In (a) the fibers are oriented normal to notch root (or crack front) and parallel to crack growth direction, in (b) parallel to notch root (or crack front) and normal to the crack growth direction and in (c) normal to notch root (or crack front) and normal to the crack growth direction.The fiber orientation (b) does not influence the fracture behaviour much, as the fibers contribute to microcracking mechanisms only and to bridging mechanisms hardly. Contrary, orientation (c) is mainly responsible for bridging mechanisms becoming effective. The energy consumption is the higher, the higher the process zone is and the higher pull-out forces and fiber pull-out displacements are. Most effective in reducing the fracture energy during biaxial loading are the fibers oriented like (a). They support the splitting procedure by forming a crack path under low energy consumption, as the adherence force between fibers and matrix is usually weak, so that an applied compression load leads to delamination. This pre-determined crack path then is the reason that a slender process zone only is formed and the fibers in the (c) orientation are not pulled much, so that the material breaks at relatively small CMOD’s already. Long fibers with a large cross section favour these mechanisms. In [15] steel fibers with rectangular cross section have been used as reinforcement and have led to an even larger decrease of the fracture energy under biaxial instead of uniaxial loading than in the present paper. The discussed model has been experimentally studied on Polypropylene FRC in [14] and on steel FRC [15] with different fiber lengths and volume contents and has been theoretically investigated by computer simulation in [16]. For more details see these three publications.

The increase of the fracture energy for FRC in the range $20\% \leq \sigma_1/f_c \leq 60\%$ appears to be cyclic and relatively small. This increasewith increasing biaxiality may be explained with the biaxial loading fracture model for plain concrete and FRC, as described in [14, 15, 22, 26]. The shallow maximum is caused by the more slender process zone under biaxial than under uniaxial loading and in addition by higher “bridging effects” of aggregates and fibers, which need higher separation and pull-out energies owing to the effective compression forces.

Many publications exist on the size effect in plain concrete, but no studies exist on this effect in FRC. If the specific fracture energy is used to roughly estimate the size of the process zone, the characteristic fracture values seem less critical under biaxial loading of FRC, owing to the smaller Gf values, than under uniaxial loading. The authors however are aware of the fact, that more research work is necessary to quantify the size effect of FRC.

CONCLUSIONS

Fracture tests have been performed with a new wedge splitting procedure under uniaxial and biaxial loading of unreinforced and Polypropylene-, steel- and glass fiber reinforced medium-strength concrete using cubic specimens with deep notches. The following re-sults were obtained:

1. The nominal notch bending tensile strength σ_{2NBS} of unreinforced concrete and FRC does not differ much under uniaxial loading conditions. The notch bending tensile strength values of FRC decrease however under biaxial loading in contrast to plain concrete in the range of σ_1/f_c=10–60% by approximately 10 %.

2. The resistance against crack propagation, which is characterized by the specific frac-ture energy, is increased by reinforcing Polypropylene fibers by a factor of 10, by steel fibers by a factor of 6 and by glass fibers by a factor of 1.5, in comparison to plain con-crete. For biaxial loading in the range of σ_1/f_c=20–60%, the Gf values of Polypropy-

lene FRC are 4.5 times, of steel fibers 1.7 and of glass fibers 1.2 times higher than for plain concrete. The results demonstrate that the specific fracture energy is much reduced under biaxial loading in comparison to uniaxial loading.

3. For the practical engineer it is important to know, that characteristic fracture values of FRC, which have been determined under uniaxial loading, cannot be used to calculate biaxial values (as it is done quite often), even if safety differences are assumed. In order to obtain reliable values, biaxial fracture experiments have to be performed.

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DESIGNING STRUCTURES USING NON-FERROUS REINFORCEMENT

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ABSTRACT. Fibre composite rods, using glass, carbon or aramid fibres combined with a suitable resin, can offer a viable alternative to steel reinforcement. The paper describes the materials and outlines the properties of the composites. Applications in bridges, tunnels and other structures are briefly reviewed. Aspects of the design of structures reinforced with composites are considered along with details of trial structures being built as part of the EUROCRETE programme.

Keywords: Aramid fibre, Bending, Bridges, Carbon fibre, Composites, Glass fibre, Reinforced concrete, Resins, Tunnels.

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INTRODUCTION

The corrosion of embedded steel in concrete can be a major problem, particularly in aggressive environments and where workmanship is poor. Design codes specify minimum concrete qualities and minimum thicknesses of cover for given environmental conditions. However, there are many examples where the specified values have proved inadequate. Hence many approaches are currently being investigated to improve the situation. These can be briefly summarised as:

• Improving the quality of the concrete

• Protecting the steel reinforcement electrically
• Coating the steel (e.g. fusion bonded epoxy coating)
• Replacing the steel with a more durable material.

This paper deals with the last topic, describing aspects of the work of EUROCRETE, a 4-year international collaborative programme to develop fibre composite reinforcing bars for concrete and the associated methods of design.

FIBRE COMPOSITES

Fibres

Currently, the most suitable fibres for the reinforcement or prestressing of concrete are glass, carbon or aramid. Each is a family of fibre types and not a particular one. They all have high ultimate strengths (in the region of 3000N/mm^2) and stiffnesses which range from about 50kN/mm^2 to over 200kN/mm^2. They have a linear elastic response up to ultimate load, with no significant yielding. The fibres are relatively difficult to handle and anchor into concrete and hence, with the exception of one aramid prestressing system, they have been combined with resins to form composite rods or grids, as described below. Short, chopped fibres such as polypropylene, which would provide little if any additional strength, are not considered in this paper.

Resins

There is a wide choice of resins available, many of which are suitable for forming composites. The choice will depend on the required durability, the manufacturing process and the cost. Thermosetting resins are generally used but thermoplastic resins are now being developed.

Composite manufacture

The most widely used manufacturing process is pultrusion, which is used to produce a wide range of structural shapes. The fibres, which may be all of one type, or a combination of materials are supplied in the form of continuous rovings. They are drawn off in a carefully controlled pattern through a resin bath which impregnates the fibre bundle. They are then pulled through a die which consolidates the fibre-resin combination and forms the required shape. The die is heated which sets and cures the resin allowing the completed composite to be drawn off by suitable reciprocating clamps or a tension device. The process enables a high proportion of fibres to be incorporated in the cross-section and hence relatively high strength and stiffness are achieved. However, the sections have a smooth surface, which provides insufficient bond for use in concrete. Hence, a secondary process is required to improve the bond.

The properties of the composite will depend on the type and percentage of fibres used. Typically with glass the ultimate strength might be 1200N/mm^2 and the elastic modulus 40kN/mm^2.

Other manufacturing processes are being developed to form two-dimensional grids, such as the Japanese NEFMAC material [1], and even complete three-dimensional grids.

APPLICATIONS IN PRACTICE

Bridges

The widest use of fibre composites in concrete structures has been for prestressing. This is probably because changing from steel to the new materials leads to little, if any, change in the design process. Hence, a number of bridges have been built worldwide, generally with conventional steel for the unstressed reinforcement. Examples include the Ulenbergstrasse Bridge in Dusseldorf opened to traffic in 1986 [2], which, along with other bridges in Germany and Austria, is prestressed with glass-fibre composite cables. In Japan a number of bridges have been built [3], mainly prestressed with aramid or carbon composites, while in the USA and Canada [4, 5] various materials have been used.

Tunnels

In Japan, fibre composites have been used for reinforcing the sprayed insitu lining of several kilometres of tunnel [1]. Here the reinforcement, in the form of a square grid in place of the usual welded steel fabric, is only required during the early life of the concrete lining.

Other applications

Fibre composites have been used in the foundations of sensitive electrical equipment, because of their non-conducting and non-magnetic properties. Other uses include marine structures and storage tanks.

DESIGN OF REINFORCED CONCRETE

Introduction

Current design codes consist of a mixture of basic principles and simple rules of thumb. The latter are based on one hundred or so years of experience of structures reinforced with steel. The designer is given no guidance as to how such requirements should be modified when changing to different reinforcement materials. In formulating new design methods two approaches are possible. The first is to develop completely new methods and the second is to adapt the existing ones. The former requires considerable experimental data and would thus tend to delay the introduction of new materials. Hence the EUROCRETE programme is developing modifications to existing design clauses, justified by comparison with the limited data available. The resulting designs may not be the most economic use of the new materials but they should result in safe structures.

In modifying the design clauses, only those concerned directly or indirectly with the reinforcement need to be changed. Those covering general principles and those concerned with the geometry of the structure remain unaltered. The following sections cover the more significant aspects.

Bending of beams

The basic principles of the behaviour of a beam in bending, such as plane sections remaining plane, should not depend on the type of reinforcement material provided it has sufficient bond. It is likely that failure will occur by compression of the concrete and not by reaching the ultimate capacity of the tensile reinforcement.

A number of researchers have studied the behaviour of beams in bending including Nakano et al [6] who used carbon or aramid composite bars, Faza & GangaRao [7,8], Nawy & Neuwerth [9,10] and Masmoudi et al [11] who used glassfibre composites and Maruyama et al [12] who used carbon fibre composites. Figure 1 shows the test results plotted against the theoretical predictions of BS8110, using a partial safety factor of 1.0 for both the concrete and the reinforcement. It indicates that the design approach is satisfactory: a factor of safety of 1.5, a reasonable value, would be required to give safe design values.

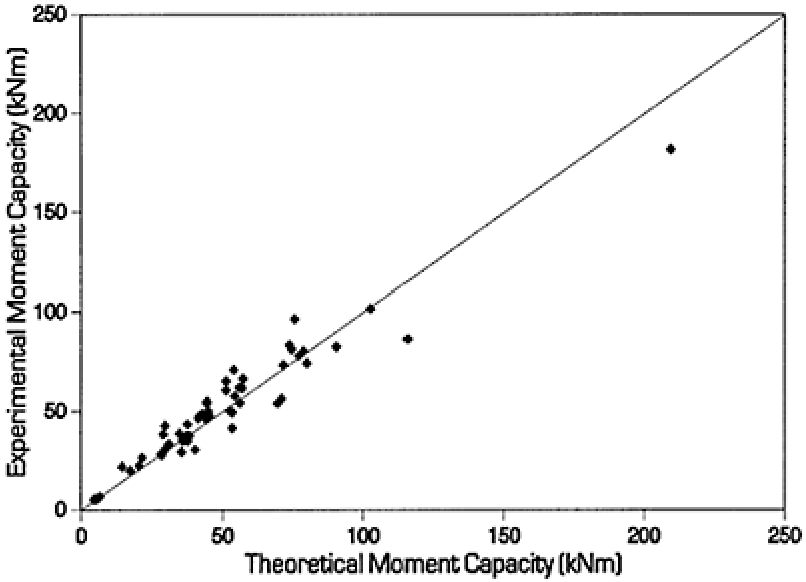

Figure 1 EUROCRETE: Comparison of moment capacities

Shear

In line with most design codes, BS8110 uses an empirical approach to determine the shear capacity of beams. The total capacity is taken to be the sum of the capacity of the concrete cross-section and that of the shear reinforcement. While this is not a true representation of the behaviour, it has been proved to give an adequate margin of safety for conventional steel reinforcement. Initially, the equations have been modified for beams with non-ferrous reinforcement as follows:

- Concrete cross-section: replace the area of non-ferrous reinforcement with the equivalent area of steel, on the basis of the modulus of elasticity
- Shear reinforcement: use a limiting strain of 0.0025 (c.f. a limiting stress of 460N/mm^2 in BS8110).

Figure 2 shows the results from Nawy & Neuwerth [9], Maruyama et al [12] and from a major series of tests by Nagasaka et al [13] and other unpublished data plotted against the predicted values again with factors of safety of 1.0. Although the scatter of results is greater than for the bending comparisons, as would be expected, the agreement is still reasonable. An overall partial factor of safety of 1.5 would lead to safe design values. Further study is required in this area.

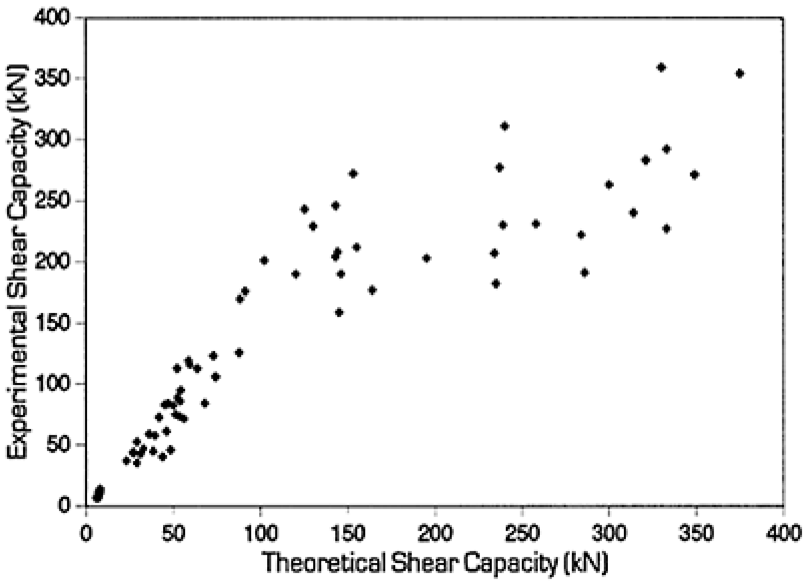

Figure 2 EUROCRETE: Comparison of shear capacities

CASE STUDIES

An important aspect of the EUROCRETE programme is the construction and monitoring of prototype structures reinforced with fibre composite bars. They can be used to identify any problems with the construction process and provide information on the longterm behaviour. The latter is a particularly useful addition to the data being obtained by other EUROCRETE participants from laboratory durability trials, both on the bars themselves and on specimens cast into concrete, and from specimens at a number of exposure sites.

Trial Footbridge

EUROCRETE was invited to build the first footbridge in Britain, and possibly in Europe, using glass fibre composite reinforcement [14] at Chalgrove in Oxfordshire. The bridge slab, measuring 5.0m×1.5m× 0.30m, was precast using Grade 40 concrete by Tarmac Precast. The bars, which were produced by GEC, were of 13.5mm diameter and were used as mesh reinforcement with rods placed orthogonally at 150mm spacing on the top and bottom surfaces. Prior to casting, a total of sixteen vibrating wire gauges were placed throughout the slab such that monitoring of the inservice strains could be carried out. After casting, fibre-optic cables were placed at soffit level over the central 2m of span which also allow inservice behaviour to be measured.

The bridge slab was transported to site by Laing Civil Engineering and placed on mass concrete abutments, using Neoprene pads as a bearing surface. Glass fibre composite handrails were fixed to the slab once it was located in position. Load tests were then carried out to 125% of the design load in accordance with BS8110 using a series of steel deadweights. The results justified the design assumptions. In addition, some dynamic tests were carried out. It is planned that these load tests will be repeated. Continuous monitoring is being undertaken electronically by the recording of a full set of data once each hour. The completed structure is shown in Figure 3.

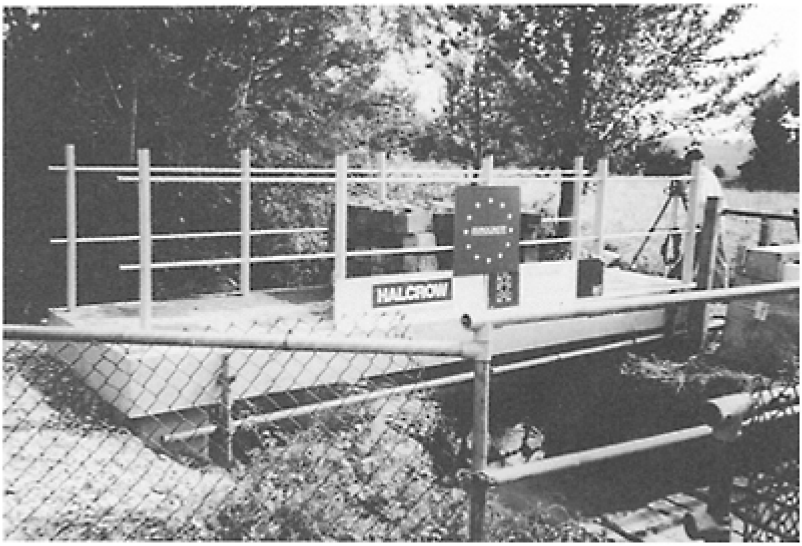

Figure 3 EUROCRETE: Trial Footbridge in Oxfordshire

Cribwall units

Cribwalls consist of small precast units which form a series of interlocking structures into which suitable free draining granular materials are placed. The result is a near vertical retaining wall. There is a growing interest in this form of construction for schemes such as motorway widening. The units are reinforced chiefly for handling purposes; their dimensions are dictated by the requirement for adequate cover to the steel.

Units from Tarmac Precast Concrete's Kriblok system and from Phi Group's Anda-Crib system were cast with glass fibre composite rods in place of the normal steel. They were tested at Sheffield University along with conventionally reinforced units and proved to be very satisfactory. It is planned that a trial length of wall will be built by Tarmac Precast consisting of a vertical strip of units reinforced with glass fibre rods with conventional units on either side.

Other structures

As part of the EUROCRETE programme, the construction of other trial structures using non-ferrous reinforced concrete is planned. These include the following:

- a 40m length of post and panel fencing required to have non-magnetic properties
- rock anchor spreader plates, for use with a non-ferrous rock anchoring system
- concrete cladding units
- coastal protection units
- a gravity fender unit in a marine location in the Middle East.

CONCLUSIONS

The trial structures being built as part of the EUROCRETE programme are demonstrating that fibre composite rods can offer a viable alternative to steel reinforcement in concrete. Safe structures can be designed using modified versions of the current codes.

ACKNOWLEDGEMENTS

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A PRELIMINARY INVESTIGATION INTO THE COMPARATIVE PERFORMANCE OF FRP AND STEEL REINFORCEMENT IN MEDIUM AND HIGH STRENGTH CONCRETE

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ABSTRACT. This paper presents the preliminary findings of an investigation into the performance of medium to high strength concrete beams containing comparable areas of fibreglass-reinforced plastic(FRP) and high tensile steel reinforcing bars. Failure of the steel reinforced beams was predominantly in flexure for the more lightly reinforced sections and in shear-bond for those with the largest areas of reinforcement whereas the FRP reinforced beams exhibited mostly shear-bond type failure with the exception of the most lightly reinforced high strength concrete beam which failed in flexure. In general, increasing concrete strength had a marginal effect upon ultimate load capacity. The development of the concept of a 'performance quotient' is proposed as an efficiency comparator for elements of different types and composition.

Keywords: Fibreglass-reinforced plastic reinforcing bars(FRP). high tensile steel, medium strength concrete, high strength concrete, flexure, shear-bond, performance quotient.

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INTRODUCTION

The current interest in non-ferrous reinforcement for use in concrete has occurred mainly as the result of the extensive problems associated with the degradation of existing buildings and structures resulting from the corrosion of steel rebar in adverse service environments[1]. Design practice for enhancing the durability of reinforced structures involves matching the assessed exposure with an appropriate amount of protection in terms of concrete grade and cover to rebar [2], [3]. This often produces a concrete with a performance potential in excess of that required for sustaining imposed service loads.

The use of fibre-reinforced polymers in concrete is not new[4] but improved manufacturing techniques and modified morphology have enabled a more efficient use of materials to produce elements which in strength and specific energy relative to performance excel that for ferrous reinforcement [5], [6]. Furthermore the reinstatement or strengthening of structures may provide another opportunity for the use of FRP in lieu of steel [7], [8]. Although, as with steel, at elevated temperatures and during exposure to fire, loss in structural integrity of FRP reinforced concrete elements could occur mainly because of the susceptibility of the resin component in the rebar.

This paper presents results from the first stage of an investigation into the behaviour of medium to high strength concrete beams reinforced with FRP and high tensile steel rebars and examines their comparative behaviour in the context of models currently used for conventional reinforced concrete design in the UK.

EXPERIMENTAL DETAILS

Materials

Concrete:—Portland cement (42.5) with uncrushed fine aggregate(M sand) and coarse aggregate (20–5mm gravel).

Reinforcement:—Steel and FRP reinforcement details are contained in Table 1

Table 1 Specified Reinforcement Details

REBAR TYPE	DENSITY	STRENGTH	ELASTIC MODULUS	BAR SIZES
	kg/m3	MPa	GPa	mm
High Tensile Steel	7,850	460 (Yield)	210	8, 12, 16
FRP	2,045	695 (Ultimate)	50	8.8, 13.2, 16.4

Concrete Properties and Mix Proportions

Details of grades, mean strengths, W/C ratios and mix proportions are shown in Table 2.In addition, all concretes had slumps within the range 30–60mm.

Table 2 Concrete Properties & Mix Proportions

CONCRETE GRADE (Nominal)	MEAN STRENGTH MPa	W/C	MIX PROPORTIONS W	C	FA	CA
C40	59	0.39	155	400	735	1170
C60	75	0.29	175	615	415	1250

Manufacture & Curing of Test Specimens

Two concrete grades(C40 and C60). three Bar sizes and and two rebar types (steel and FRP) were investigated giving a total of 12 No. mixes for all combinations. Details of test specimens manufactured and curing are given below.

Test Cubes

3 No. 100mm side test cubes were manufactured from each mix and cured adjacent to the test beam in the laboratory. The results are in Table 3. column 7

Beams composition

One beam 103mm(breadth)×203(depth)×2.740mm(length) containing 2 No. steel or FRP bars was manufactured from each mix; further details shown in Figure 1.

Testing of Concrete Specimens

Cubes were tested at 42/43 days, and compression tests were carried out in a 3000kN capacity compression testing machine in accordance with BS1881 Part 116.

Beams were tested 42/43 days in four point bending in stroke control mode. A displacement transducer placed at mid-span enabled a continuous record of load versus displacement to be made. In order for failure mode transitions to be observed as area of rebar(A_r) and concrete grade were changed no specific shear reinforcement was included. Details and dimensions of testing configuration are also shown in Figure 1.

FAILURE MODELS AND MECHANISMS

Models

In general these follow the basis of design for steel reinforced members in BS8110 with minor variations and are summarised below with appropriate explanation (3).

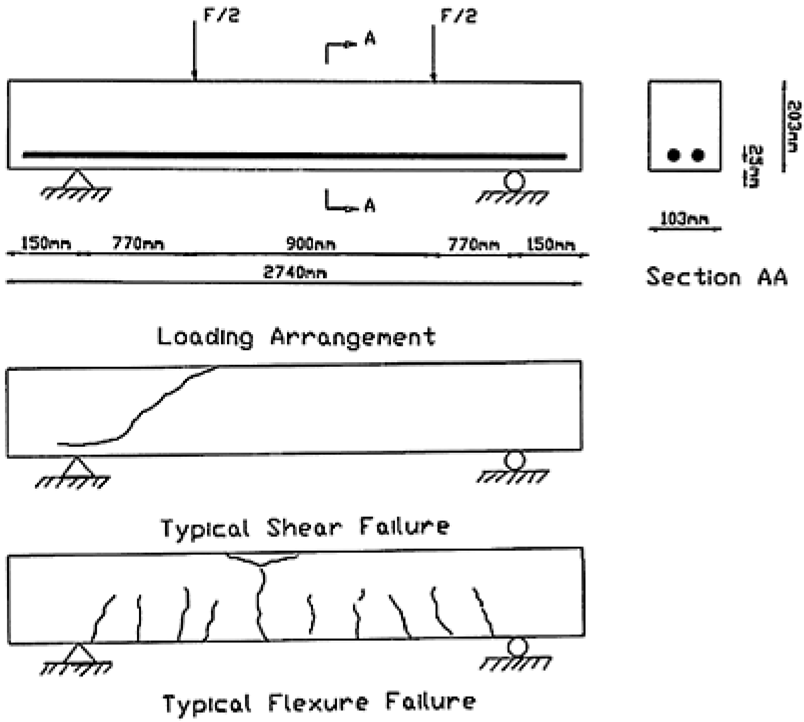

Figure 1 Beam Dimensions and Failure Modes

Ultimate Bending Moment(M_u)

The ultimate theoretical bending moment(M_u)=Cz=T *z*, where

Force in concrete stress block (C)=0.67f_c b d 0.9x, Force in rebar (T)=f_v A_s Lever Arm (z)=(d−0.45×), Depth to rebar(d)=178mm, Beam width(b)=103mm

Neutral axis depth(x) & Lever Arm(z)

For flexural failure in an under-reinforced section then at ultimate load and equating forces in concrete stress block (omitting materials factor, γ_m) with those in steel at yield gives:

$0.67f_c bd0.9x=f_y A_s$ hence

$$x = \frac{f_y A_s}{0.603 f_c\, b\, d\, 0.9}$$

thus

$$z = d - \frac{f_y A_s}{0.603 f_c\, b\, d\, 0.9}$$

Therefore the the ultimate moment(M_u) may be calculated from concrete cube strength(f_c) rebar strength(f_y) and area of rebar(A_s). It should be noted however that when applied to FRP where area of rebar is A_f this model equation is likely to give an underestimate of the moment capacity as there is minimal yield of this material at failure.

Resistance to Shear (vs)

After BS8110 (cl 3.4.5.4, Pt 1), but omitting the materials factor (γ_m). is expressed as follows:

$^{v}s(max)=k_l\, k_2\, 0.79\, (100\, A_r/(b\, d)^{1/3}(400/d)^{1/4}$

where k_1=1.0 and $k_2=(fc/25)^{1/3}$ where fc not>40MPa

Failure mechanisms

Sketches of the two principal failure mechanisms are shown in Figure 1. Initially vertical cracks emanating from the bottom surface within the middle section preceded failure in all cases. In some cases these were followed by inclined cracks towards the supports. At the ultimate condition beams failed in two alternate modes.

Flexure failure:—resulted from yielding of the steel or sudden rupture of the FRP followed by crushing of concrete in the compression zone within the mid-span section The actual ultimate moment (M_u') is calculated on the basis of the maximum load at failure(F in kN) its lever arm 0.770m and takes into account the self weight of the beam(kN) and span(m) which approximates to:

$$M_u' = \frac{\text{Failure Load} \times \text{Lever Arm}}{2} + \frac{\text{Beam self-weight} \times \text{Span}}{8}$$

thus; M_u'—0.385 F+0.40 kNm

Shear and Shear-Bond failure:—namely Type I resulted from simple diagonal shear between load point and support and Type II by shear-bond failure; along a line emanating from a loading point and passing diagonally across the section then horizontally along the rebar-concrete interface between one loading point and a support (9). The average failure shear stress across section(vs) was taken as approximately:

$$vs = \frac{1000\,(F + \text{Beam self-weight})}{2\,b\,d} = 0.545(F + 1.3)\ \text{MPa}$$

Performance Quotient (Qp)

To assess the efficiency of the beam in resisting load it is useful to relate cost (in real terms or energy equivalence) to strength(6). As an alternative to obviate the effect of capricious influences associated with market and manufacturing factors which vary from time to time, the authors propose a quotient relating the load capacity with the load bearing potential based upon a measure of the strength of component materials and their average cross-sectional area throughout the beam. This may be expressed as:

$$Qp = \frac{1000\,(F + 1.3)}{f_c\,(b\,d - A_r) + f_r A_r}$$

where A_r = area of
rebar

f_c = concrete strength

f_r = rebar strength

RESULTS AND DISCUSSION

Table 3 contains details of the beam references and experimental programme together with rebar details in columns 1–4. Results from the experimental work and those from the theoretical models are contained in columns 5 to 14.

Failure modes

All steel rebar beams failed in flexure except those with 16mm diameter rebars(TB9 & TB11—shear-bond mode) whereas FRP rebar beams failed in shear-bond with the exception of C60 beam with 8.8mm diameter bars (TB8—flexural rupture of bars) and C40 with 8.77mm diameter bars (TB6—simple shear). See column 5.

Failure Load

Failure loads suggest that the steel rebar beams were more sensitive to rebar area and less sensitive to concrete grade than those reinforced by FRP, however cognisance must be taken of failure mode in this respect. See column 6.

Actual and Predicted Failure Mode and Capacity

The design models for steel rebar beams were consistent with the test results including the transition from flexural to shear failure as area of rebar was increased (see TB5, TB1 and TB9). When applied to FRP beams these overestimated shear capacity but underestimated flexural capacity (TB8); the latter finding contrasts with some recent work by Brown and Bartholomew [10]. See colunms 5, 6 & 10–13.

Performance Quotient

For flexural failure the Performance Quotient for steel and FRP reinforced beams (TB5 & TB8) were comparable although not for the shear type failures. See column 14.

LOAD VERSUS DEFLECTION BEHAVIOUR

Load versus deflection behaviour for C40 concrete with 8mm steel rebars (TB5) compared with C60 concrete with 8.8mm FRP(TB8) rebars are shown in Figure 2. This suggests that although the FRP reinforced concrete displays a higher deflection at a similar failure load, the toughness of the ‘FRP’ beam (indicated by the area under the Load vs Deflection curve) is much higher than that for the steel beam of a similar performance quotient for flexural failures.

Table 3 Summary of beam properties and performance

< Column Number >													
1	2	3	4	5	6	7	8	9	10	11	12	13	14
Beam Code	Concrete grade	Reber No. Dia/ Type	Area Rebar mm2	Failure Mode	Failure Load kN	fc Cube Str. MPa	NA Depth mm	Lever Arm mm	Mu' Actual kNm	Mu Theor kNm	vs Actual MPa	vs(max) Theor MPa	Perform. Quotient Actual
TB5	C40	2–08S	101	Flexure	24	61	12	173	9.64	8.01	0.57	0.93	0.01863
TB1	C40	2–12S	226	Flexure	50	55	31	164	19.65	17.08	1.20	1.21	0.0411
TB9	C40	2–16S	402	Shear-Bond	68	61	49	156	26.58	28.85	1.63	1.47	0.04794
TB6	C40	2–08.77F	121	Shear	20	69	20	20	169	8.10	14.20	0.98	0.1369
TB2	C40	2–13.19F	273	Shear-Bond	34	57	54	154	13.49	29.23	0.81	1.29	0.02531
TB10	C40	2–16.44F	425	Shear-Bond	29	51	93	136	11.57	40.13	0.69	1.50	0.02216
TB7	C60	2–08S	101	Flexure	24	73	10	173	9.64	8.02	0.57	0.93	0.01574
TB3	C60	2–12S	226	Flexure	48	72	23	168	18.88	17.44	1.15	1.21	0.03044
TB3	C60	2–16S	402	Shear-Bond	58	75	40	160	22.73	44.77	1.39	1.47	0.0339
TB11	C60	2–08.77F	121	Flexure	26	72	19	170	10.41	14.24	0.62	0.98	0.01678
TB4	C60	2–13.19F	273	Shear-Bond	38	79	39	161	15.03	30.49	0.91	1.29	0.02127
TB12	C60	2–16.44F	425	Shear-Bond	28	76	62	150	11.18	44.24	0.67	1.50	0.01543

Key **Steel rebar(S)** **FRP rebar(F)**

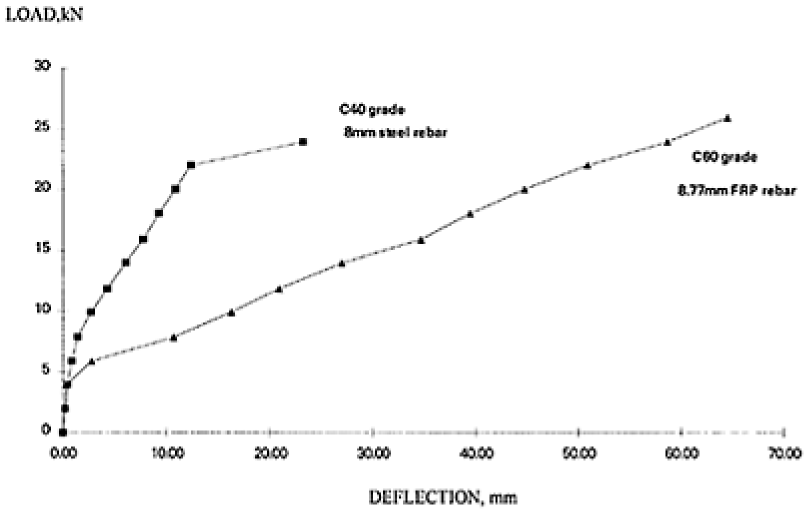

Figure 2 Load versus Deflection for
steel and FRP reinforced beams

Figue 3 shows a similar curve comparing 16mm steel rebars with 16.44mm FRP rebars for C60 concrete (TB11 & TB12) and indicates that shear-bond failure tends to favour the perfomance of the steel rebar beam compared with the FRP rebar beam.

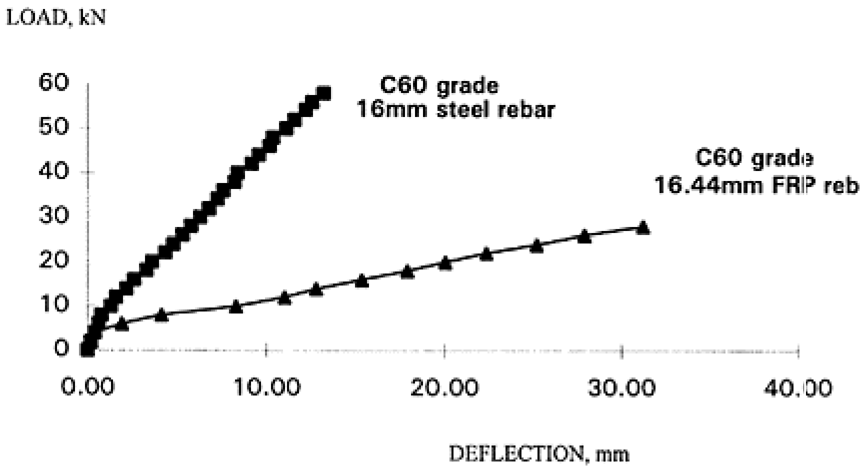

Figure 3 Load versus Deflection for
steel and FRP reinforced beams

PRACTICAL IMPLICATIONS OF RESULTS

The results suggest that performance of the FRP beams containing the higher areas of tension reinforcement could be greatly enhanced by improving shear-bond capacity hence utilising their maximum flexural strength capability. Certain properties of FRP should enable this to be attained more economically than with steel rebar. It is proposed to explore new design strategies in future stages of the investigation.

CONCLUSIONS

1. The mode of failure appears to be influenced by rebar type and area although concrete grade was a confounding factor for FRP reinforced beams.
2. Conventional design models for steel reinforced concrete may require modification for FRP rebar beams especially for shear failure.
3. For flexural failure FRP beams display a greater capacity to absorb energy than steel for similar load capacity, although they exhibit reduced stiffness.
4. A performance quotient relating load capacity to 'section potential' for similar geometry may be a useful efficiency comparator.
5. The development of a design strategy for improving shear-bond performance without significantly increasing rebar area could be applied to FRP beams to enhance their performance.

ACKNOWLEDGMENTS

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STRUCTURAL BEHAVIOUR OF BAMBOO REINFORCED CONCRETE BEAMS AND SLABS

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ABSTRACT. In Brazil, bamboo has been studied as an engineering material. One of its applications is as concrete reinforcement. Experimental studies were developed at the Catholic University of Rio de Janeiro and the Federal University of Paraiba using 3m span beams and slabs. This paper presents and discusses some of the results of these tests.

Keywords: Bamboo, Concrete, Beams, Slabs, Bamboo Reinforced Concrete.

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INTRODUCTION

Renewable materials are having crescent interest to be used for humanity. In the field of engineering, bamboo is a vegetal that presents vantages as fast growing and good mechanical properties. A comparison with other natural and engineering materials revealed a superior structural efficiency [1]. Since some years ago, bamboo as engineering material has been studied in a research programme involving Catholic University of Rio de Janeiro and Federal University of Paraiba, in Brazil. During this time, some aspects were investigated: physical and mechanical properties of bamboo species growing in Rio de Janeiro [2] and Paraiba [3–4], treatments against insects and bamboo propagation [4], bamboo-concrete bond [5], bamboo special truss [6], bamboo reinforced concrete elements [7–10]. This papers presents and discuss results from experimental tests made in bamboo reinforced concrete beams and slabs.

EXPERIMENTAL DETAILS

Materials

Two kinds of concrete were employed: normal concrete with granite coarse aggregate and lateritic concrete, with lateritic stones as coarse aggregate. The last presents lower Young module [11].

Bamboos from species "Dendrocalamus giganteus" and "Bambusa vulgaris" were utilised. Tables 1 and 2 indicates some properties of these bamboos obtained in Brazil.

Beams

Until now, eight 3 m span beams were tested. The first three beams were built in lateritic concrete. The others in normal granite concrete. The beams, instrumented with electrical and mechanical strain gages, are show in Figure 1. Different reinforcement tax were used, trying to improve beams behaviour.

Table 1—Mechanical characteristics of bamboo used

Species	Compression strength (MPa)	Tensile strength (MPa)	Flexion strength (MPa)	Young's module (GPa)
Bambusa vulgaris	65	115	131	9
Dendroc. giganteus	77	115	152	11

Table 2 Physical properties of bamboo used

Specie	Internode (cm)	Diameter (cm)	Thickness (cm)
Bambusa vulgaris	35–45	7–8	0,6–0,8
Dendroc. giganteus	55–65	12–14	1,0–1,2
B. vulgaris shard	34–37	7–8	0.6–0.8

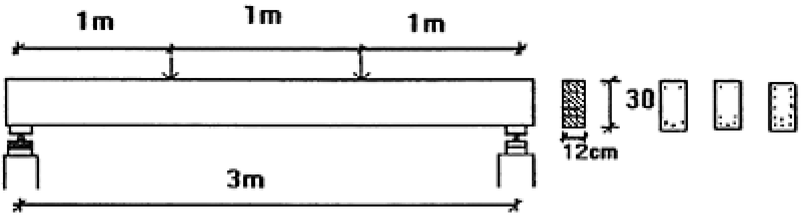

Figure 1 Tested beams

Flexure reinforcement consisted in to cut longitudinally the culms making 2–3 cm large rods. In some beams bamboo nodes were removed to make a rectangular plane bar (Figure 2a). To improve bond, the bamboo rod received superficial treatments. As although that bond were not good, the rods were employed with part of nodes. (Figure 2b). Finally, two kinds of artificial connectors were used: a little granite stone glued with epoxy (Figure 2c) and 3 cm long pieces of 8 mm construction steel (Figure 2d). Shear reinforcement were 5mm construction steel.

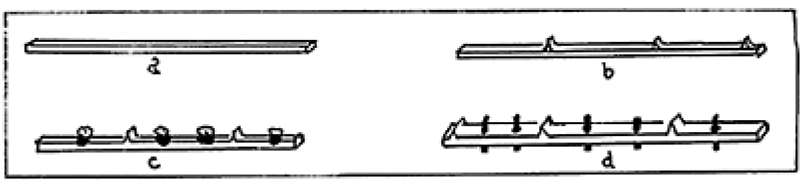

Figure 2 Bamboo rods

Bamboo-concrete bond

To study bamboo-concrete bond, some tests were made, as show in Figure 3. Despite the push out test can give only an average value of bond stress, it serves to compare the behaviour of bamboo rod treated in different way. Some treatments were tested in the rods with and without node: hot bitumen, bitumen emulsion, oil paint, epoxy. Even a steel wire was rolled up in some rod treated with bitumen.

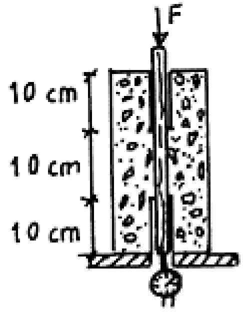

Figure 3 Bond test

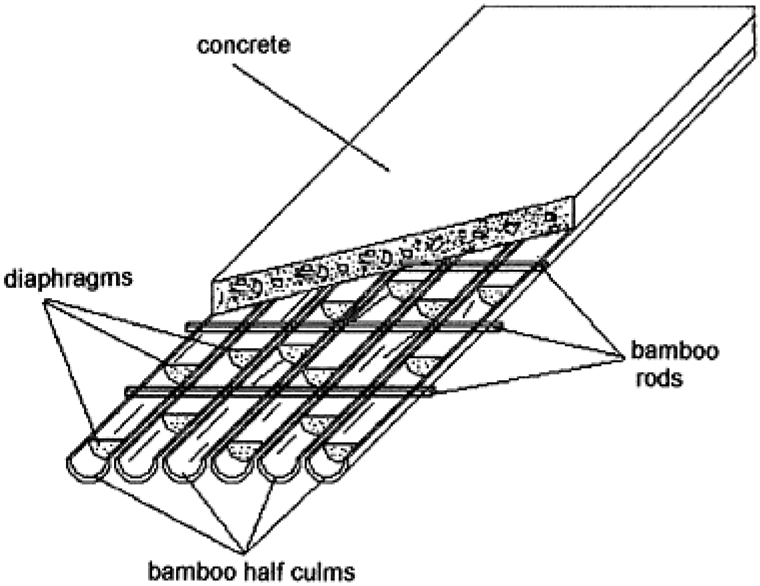

Figure 4 Permanent shutter slab.

Permanent shutter slabs

Bamboo culms are void internment. At the nodes they have a diaphragm, that has a good shear strength. This fact, conducted Ghavami [12–13] to associate the septa to connectors used in concrete-steel mists structures. Half sections of bamboo set together to the width of the slab is used as permanent shutter. Figure 4 shows this kind of slab. The first slabs were build with 120 mm high and results were so good for serviceability loads that after only 10 cm high slabs were used, including bamboo half culms. The 3 m span slabs were instrumented with mechanical and electrical strain gages as indicated in Figure 5.

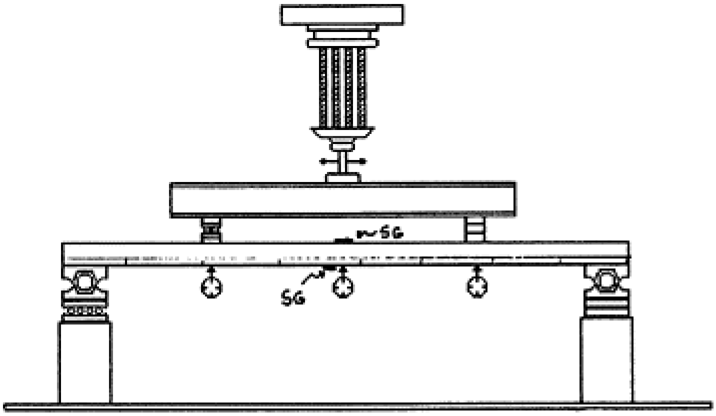

Figure 5 Slab flexion test

RESULTS AND DISCUSSION

Beams

Initial tests shown that bamboo reinforced concrete beams are more flexible than steel ones. This is due, further on bond problems, to the lower Young module of bamboo. In fact, it can be between 1/20 and 1/10 of steel modules. Then, although bamboo tensile strength is near steel tensile strength, it is necessary to put great reinforcement tax in bamboo beams. Experimentation shows ideal tax between 3 and 4 %.

Initial bamboo beams cracks presented large opening and were very spaced. A skin reinforcement was then adopted. This improved behaviour but not sufficiently as attended. The next step was improve bond using artificial connectors. Two kinds were tested separately. The first, consisted on two little stones glued with epoxy in each internode (Figure 2 c). The second, small 3 cm steel construction, also two per internode (Figure 2 d).

Figure 6 shows curves load deflexion of concrete-bamboo beams in meddle span where the two types of connectors were used. The load was applied in two cycles: first until 20 kN e after until rupture of the beam. The beam with stone connectors presented better performance. Concrete compression strength was near 25 MPa.

Figure 7 shows curves moment-bamboo strain and moment-concrete strain at the centre of the beam where stones connectors were used. The concrete strain gage was installed about 2 cm of the beam top.

If a deflexion equal to span/300 is considered as serviceability limit, then the beam reaches this limit state at a load Fs=32 kN. To produce the same deflexion, it will be necessary a distributed load 14.5 kN/m. Considering ultimate load (Fu=88 kN), it is interesting to see that the relationship Fu/Fs=2.75. Comparing with a series of beams tested, in lateritic and normal concrete, steel reinforced (b=10 cm; h=30 and 25 cm; L=3 m; As /b.h=0.64 to 0,96 %) this relationship was near 1, 8–1, 9 [14].

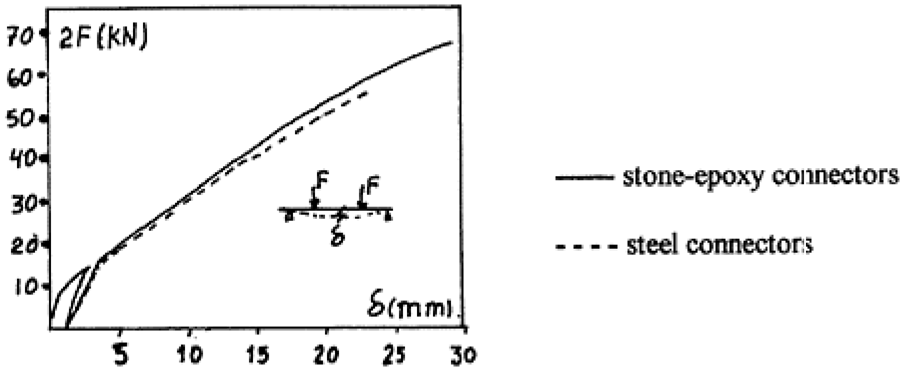

Figure 6 Curves load-deflexion at centre of beam

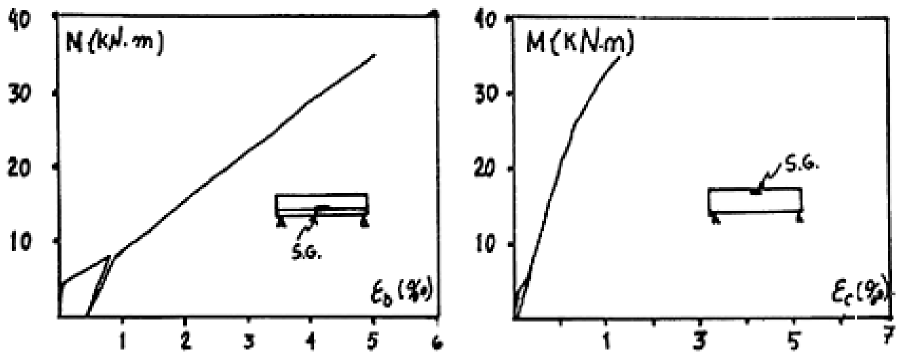

Figure 7 Curves moment-bamboo strain and moment-concrete strain at centre of beam with epoxy glued stone connectors.

Bamboo-concrete bond

Figure 8 shows some curves average bond stress-displacement, obtained as indicated in Figure 3. The treatment with products from petrol instead improves bond, it makes worse. Many other curves were achieved (with normal and lateritic concrete) showing the same fact. Even the procedures to roll up the bamboo rod with a wire, or pressing treated rod against sand to give rugosity to the surface, are not very efficient. When epoxy was used, it was impossible to push out the rod. But it was not used in beams because its cost. Other curves taken with artificial connectors are been obtained.

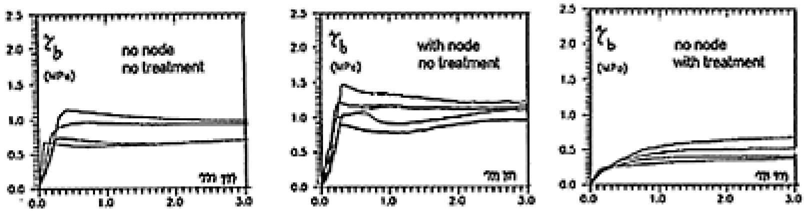

Figure 8 Curves bond stress-displacement of bamboo rod

Slabs

Figure 9 shows load-deflexion, moment-concrete strain and moment bamboo strain for the first slab tested. Load was applied in cycles.

For small load level, using Strength of Materials formulas it is possible to transform concentrate loads in distributed loads that produce the same deflexion. Considering the serviceability limit attained when meddle span displacement is equal to L/300, table 1 shows equivalent distributed load, bamboo strain and concrete strain at this displacement. It is possible to see that even in the worse case (L3) the distributed load (6.7 kN/m^2) is biggest than normal accidental charge for common slabs (1, 5 to 2 kN/m^2). Even considering revetment load and creep effect for self weigh and revetment load, the serviceability limit is not reached. At this load, Materials present still little strain.

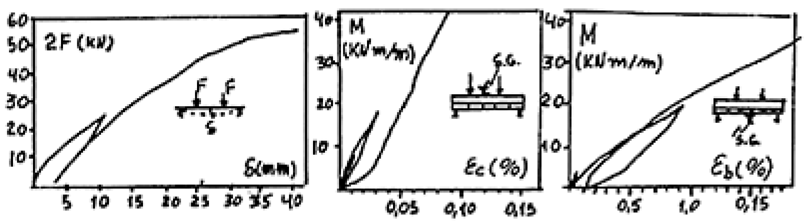

Figure 9 Load-deflexion, moment-concrete strain and moment bamboo strain of slab

Table 1 Results from slabs experimentation

slab	S1	S2	S3	SL1	SL2	SL3
concrete strength (MPa)	40	38	22	23	21	25
breadth (cm)×height (cm)	12×63	12×69	10×60	12×63	12×63	10×64
load at L/300 deflexion (kN/m^2)	17.1	13.1	6.7	12.5	10.1	8.7
concrete strain (%)	0.031	0.029	–	0.058	0.046	–

bamboo strain (%)	0.094	0.065	–	0.069	–	–

S—slab in normal concrete, SL slab in lateritic concrete

The load employed (Figure 5) produces pure bending in the central third part of slab. In all slabs, the rupture started by an inclined crack near the load, where there are normal and shear stress (Figure 10). At this time, the slab is already very deformed.

Depending upon shear stress value, its possible to distinguish three phases when load is increased: the fist, when load is small, bond for friction and adhesion guarantees that the two material work together; the second, when shear stress at contact bamboo-concrete area is greater than friction stress, the slab integrity is warranted by the diaphragms. As they have some flexibility, they allow small relative displacements..

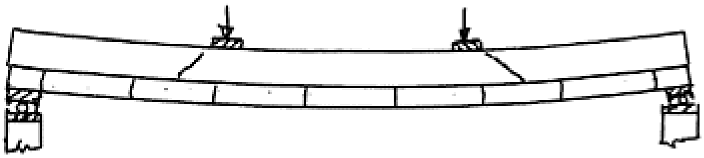

Figure 10 Crack near ultimate load

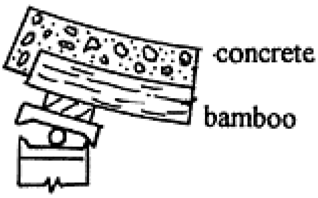

Figure 11 Displacement between concrete and bamboo at slab extremities

Finally, when diaphragm shear strength is surmounted, these relative displacements become visible (Figure 11). Interesting to note that even when there is a big displacement between concrete and bamboo at slab extremities, it can still receive load. The final rupture arrives when inclined crack reaches the slab top. The deflexion are really very big.

There is a factor near two between the maximal load and that one that produces deflexion equal L/300.

CONCLUDING REMARKS

The use of bamboo as beam reinforcement is feasible since some precautions are considered. Skin reinforcement is necessary. Main reinforcement tax must be between 3,5 and 4,5 %. Bamboo treatments with petrol products are not advisable in Northeast of Brazil, where temperature is always high. Artificial connectors increase load capacity of beams and improve its behaviour.

Tests shows that bamboo permanent shutter slab is a promising structural element where this vegetal is abundant.

Under serviceability loads, permanent shutter slabs have a behaviour more then satisfactory. Even with 10 cm height, limit state is not reached for normal accidental charges.

Under serviceability loads materials strain are small.

Maximal load is reached with large displacement that prevent its imminence.

There is a large safety factor between rupture and serviceability load.

It is possible to think about an artificial connector for the slabs when bamboo internode is very long. Some laboratory experiences was already started.

At the Structural Laboratory of Federal University of Paraiba, a 10 cm height slab is loaded with sand, giving a 2,5 kN/m^2 load, since September 1993. The central deflexion are been observed to have an idea about bamboo concrete slab under long time load. Until now, creep displacement is about twice elastic one. The slab is in a very good condition.

To large use of bamboo reinforced concrete structural elements, it is necessary to study better how to guarantee durability of bamboo into concrete environment. Until now, observations made in Brazil with bamboo immersed in concrete for 6–7 years show that it is in good conditions.

In Brazil Northeast, the only insect that attacks bamboo is a Coleopteran “Dinoderus minutus”. The treatment more effective against it consists to put bamboo culms or half culms immersed in diesel oil.

Bamboo concrete structural elements are feasible to be used in rural constructions in development countries where this vegetal is available.

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Theme 6
DESIGN AND CONSTRUCTION

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Technical University of Brno
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Transport Research Laboratory
United Kingdom

Leader Paper

Design and Construction

Professor T P Tassios

National Technical University of Athens
Greece

DESIGN AND CONSTRUCTION

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ABSTRACT. It is first recognised that the production of a “structure”, from its conceptual design up to its maintenance, is a continuous process; splitting into Design and Construction stages is too rough and occasionally dangerous. The paper attempts to identify the two way information streams between these two parts of the entire technical process. To this end, several “construction dependent” pieces of information needed for a final analytical design are inventorised. Moreover, even at the preceding stage of the conceptual design important construction-governed data should be used in order to check the validity of that design, as early as possible.

The paper ends by enumerating some of the design-like operations during the construction stage. In conclusion, it seems that optimisation is reached if these interactions are recognised, formalised and facilitated.

Keywords: Design, Construction, Conceptual Design, Concrete properties, Maintenance, Management.

Professor T.P.Tassios is Director of the Reinforced Concrete Laboratory, Nat. Tech. University, Athens, GR. He served as President of CEB, chairman of the CEB/FIP Model Code 90, and chairman of working groups for the EC8. He is the author of 250 papers and books in several languages. He also served as Consultant in concrete technology, bridge design, repair and strengthening, and durability.

INTRODUCTION

For centuries, the structural engineering process was an entire and uninterrupted entity: The Engineer (one and the same person, the “Master-Builder”) was responsible for the conception the “plans” (very general though) and the execution; Design and Construction stages were not yet separated. Besides, most of the design was effectuated during construction, following the method “build and see” (or rebuild). Consequently, omission of instructions, and erroneous information from the “designer” to the “contractor” did not make sense; similarly, all feeding-back information (regarding actual conditions in the job site) was spontaneously collected by the Master—Builder. Thus, no leakage of responsibilities could ever be observed at the interface.

Since the entire production process was (more or less artificially) splitted into a “Design” and a “Construction” stage, we tend to think that these stages are in fact separated and that only a “one way” influence remains, i.e. from design to construction.

Everyday experience, however, shows that the structural process is always a real continuum; “two ways” interactions are manifested within it. And it is expected that the entire process will be optimised if all these interactions are carefully identified, sincerely recognised and facilitated. If not, some characteristic failures may (rather frequently now) remind us that we cannot disregard the nature of things: A designer “proud of his FEM package” is still responsible for

- the unbuildability of crucial column-beam joints (because of congested reinforcement),
- the cracking of unaccessible slabs, since intensive shrinkage would not be prevented by means of (expected?) curing, and
- the inadequate level of prestressing of long tendons, because of the absence of appropriate specifications regarding relevant “friction—reducing” techniques.

For each of these failures, some designers could answer “since my analysis is correct, all this is not my responsibility, the Contractor should have thought of it”. But good designers, keeping the spirit of the ancient Master Builders and recognising the continuum of the structural process, could have prevented these failures by means of an appropriate re-dimensioning, additional skin-reinforcement or specific recommendations, respectively.

Nevertheless, in a “total quality environment, early identifications of such potential risks should” be a “must” for the Contractor as well; he should also be inspired from his ancestors the Master Builders, and act accordingly towards an “upstream” care taking.

This is the kind of issues this paper intends to analyse. In Engineering it is not a good intention or an elegant instrument (be it a mathematical solution or a piece of sophisticated construction equipment) that really counts: The final engineering product only matters; and what we are going to discuss here, may equally contribute to the targeted serviceability and safety of this final product the same way as any advanced analysis may do.

More specifically, this paper will address the Design/Construction interface problems appearing in Concrete Structures; maintenance will also be considered.

And in doing so, we will bear in mind the needs of young Engineers; some of them, under the fashionable land slide of computerised techniques, may tend to forget the amplitude of our professional duties.

CONCEPTUAL DESIGN

The design process starts always with a conceptual stage; To serve a desired purpose, the Engineer conceives a first (be it rough) structural scheme; experience and imagination help him in this creative first step. Most of the characteristics of the final structure are already (and apriori) decided at this stage! Analytical verifications which follow, may easily “approve” different solutions although some of them may not be feasible (i.e. their doubtful construction or their impossible maintenance may eventually jeopardise basic performances).

In other words, since “buildability” and “maintainability” are not conventionally checked by means of relevant provisions of Codes, these basic characteristics should be confirmed otherwise: It is precisely within the conceptual design that those “verifications” should be carried out. And this is already a crucial contact between Design and Construction and Maintenance; that is why it is difficult to be a good designer without being an acceptable constructor.

Technical Availability

There is an understandable technocratic tendency in every Specialist; design Engineers could not make exception: Each new design-case may be an opportunity for a modern and impressive achievement, which may publicise the capacities of the designer (and make him rich, in glory at least). Independently of any moralistic criticism such an attitude should receive, it is clear that “modernity” and “impressiveness” will not necessarily optimise the final performance of a structure:

Available techniques and workmanship

If the structural “solution” conceived cannot be easily implemented by available contractors, there is a higher risk of gross-errors which may disproportionately (and unknowingly) increase the probability of failure. It is interesting to note here that our sophisticated theories on Structural Reliability are absolutely unable to operate in such an environment of abnormally high risk of gross-errors.

A HYPAR shell on top of that mountain may be an “elegant” solution, indeed, but locally available contractors as well as the additional instability of formwork produced by strong winds, should possibly discredit that specific structural concept.

Available materials

The weaponry of concrete technology became quite rich nowadays: Higher performance concretes, polymer modified concrete, fibre reinforced concrete, shotcretes, etc., offer a variety of solutions to serve specific requirements. Similarly, considerable development in reinforcements is made: Completely weldable steel, corrosion protected bars, smart splicing devices, external prestressing, versatile steel-concrete composites, or industrialised fixing elements. Without mentioning the various protective materials (from special grouts and paintings, up to curing compounds), it is clear that a designer keen to offer the most appropriate solution, should be familiar with these developments and make a more imaginative selection than just “reinforced concrete” of a given class of strength (only!). To this end, modern designers should be educated (intra—or extramural education) in fields broader than mathematical modeling alone. Once again, a good designer should be an acceptable Material Technologist.

Anyway, it will be very much appreciated if the design of a public building in a region poor in strong aggregates, would not be based on a high strength prestressed concrete…

Economy in Construction

The obvious basic requirement of economy is supposed to inspire even conceptual design.

However, economic solutions can only be proposed if the principal designer has a thorough knowledge of construction techniques and their real costs rather than just “unit prices”. As an example, the issue Factory vs. Site production may be irreversibly influenced if an initial conceptual design was in-situ minded; one cannot successfully “translate” a monolithic structural concept into a precast one.

Anticipated Quality Assurance

Normally, every designer should be entitled to know in advance the level of the quality assurance which will be secured during the construction stage. The use of (economically and functionally justified) sophisticated structural solutions can only be encouraged if a high level of a quality assurance scheme is envisaged. Otherwise, conservatism should prevail, and less advanced (eventually more expensive) solutions should be adopted. Owners may find here an incentive to seek global economy by envisaging in advance higher quality assurance levels.

A typical example may be the case of an underground technical work exposed to a medium aggressive sulfate environment: The use of sulfate resistant cements (with some complication in the job-site, together with an increased risk vs. chlorides) may be avoided by adopting an OPC concrete with a water cement ratio of the order of 0.35, provided that an appropriate quality control will pragmatically insure the desired low permeation level everywhere.

Anticipated Maintenance Level

Here again, the designer seems to be unprotected from the owners he is invited to serve: Owners very seldom specify the kind or the level of maintenance they are going to impart to their property. Besides, owners and users may frequently change during the life span of the structure we are going to design today; occasionally, even physical/chemical environments may change in our industrial world. With such a rather gloomy perspective, some more economical structural solutions cannot be proposed. Progress is hindered because of this uneven distribution of rationality in the socioeconomic environment where we live.

Designers have therefore an interest to follow a multiple approach on this matter:

a) They may request written statements from their client on the subject, if the importance of the structure may justify such an approach.
b) Otherwise, designers should rather assume that maintenance will be rudimentary, and they should set forth structural concepts less sensitive to in-time alterations—although such an approach may deprive us of the pleasure and the economy of some more elegant solutions.
c) Nevertheless, a good designer seeks always structural solutions enhancing inspectability and avoiding unfavourable microclimatic consequences.

d) Last but not least, it is recommended to add on drawings and in specifications an explicit warning about declining responsibility in case any future user alters any structural member or any crucial secondary part (e.g. infill walls).

Some examples should illustrate this important issue.

- In a modern swimming-pool, the designer was assured that a complete ventilation system would constantly bring down the relative humidity of the air. What finally was brought down, was the ceiling hanging from the main structure by means of naked steel bars (arranged in an non-inspectable area); their corrosion produced the failure.
- Compared to the famous accident of the Berlin Congress Hall this failure differs only in scale. In both cases, however, it is not easy to declare the designer non-guilty…
- In some seismic-prone regions of Europe, failures of columns were observed because independent electricians had brutally incorporated an electric switch- board into a R.C. column…

ANALYTICAL DESIGN

By means of appropriate criteria (application of analytical models and or of practical rules), the structure initially conceived and temporarily dimensioned shall now be checked versus several Serviceability and Ultimate limit states; I will not miss the opportunity to praise the rational possibilities offered to the designer by the new generation of Codes (and much better by the CEB-FIP Model Code '90) to check separately and conveniently any structural and functional requirement needed, on a case by case basis.

In doing so, however, the designer should once more take into account the real construction conditions or, preferably, to specify explicitly the corresponding conditions, thanks to which the relevant assumptions made in design will be implemented. In the subsequent paragraphs, an attempt is made to inventorise some of those assumptions, together with occasional quantification of the consequences of the naive equation "Design=Analysis".

Concrete Quality

All the adjectives we use to describe our concrete structures (plain, reinforced, prestressed, precast, projected etc.) cannot reduce the fundamental importance of concrete itself. For the benefit of their final products, designers should understand about "concrete" something more than just "tabulated" values of its mechanical properties.

Compressive strength

It is rather instructive to recall here the two basic handicaps of the very concept of "characteristic compressive strength", f_{ck}, of concrete:

First, we are all bound to the probabilities accepting a defective lot because of the economically imposed O.C. lines (related to the compliance criteria).

Second, all we know about that strength comes from laboratory made and cured specimens; consequences of transportation, placing, compacting and curing in-situ are not measured! (Sometimes, cores taken from the bottom of a column and from the top of a slab, both well concreted and cured, may legitimately differ by more than 50% in strength).

The combination of these two handicaps of the concept of “characteristic strength” should at least remind us what an interest we have as professionals to give further emphasis to the very details of the construction stage, on which a considerably larger part of the real safety is depending upon.

Besides, one of the best ways to secure low creep effects is to postpone striking of falsework, as it should be clearly requested by the design documents, after appropriate calculations if needed.

Concreting programme

There is no excuse for omitting a thermal analysis (in favour of, say, a dynamic analysis). Medium massive building-elements under adverse ambient conditions may be drastically cracked after concreting, because as designers we may have decided that this is a “construction problem”, although the selection of concrete pour areas and their sequence is nowadays the subject of intelligent softwares.

Similarly, uncracked R.C. walls are rare; hygric and thermal shrinkage hindered at the foundation level, is a “construction” problem inviting for drastic design provisions.

Curing

This is a decisive construction operation which, despite its numerous structural consequences, is not remunerated separately. Its importance in design is shortly reminded hereafter.

The disproportionate scattering of shrinkage values (of real concretes in-situ) should be beared in mind of every designer; otherwise, the deterministic “calculation” of an acceptable distance between consecutive permanent joints may lead to unpleasant surprises of unexpected cracking. These phenomena may be further accentuated, through the decades, because of an ill-defined tensional fatigue of concrete under cyclic hygrothermal conditions.

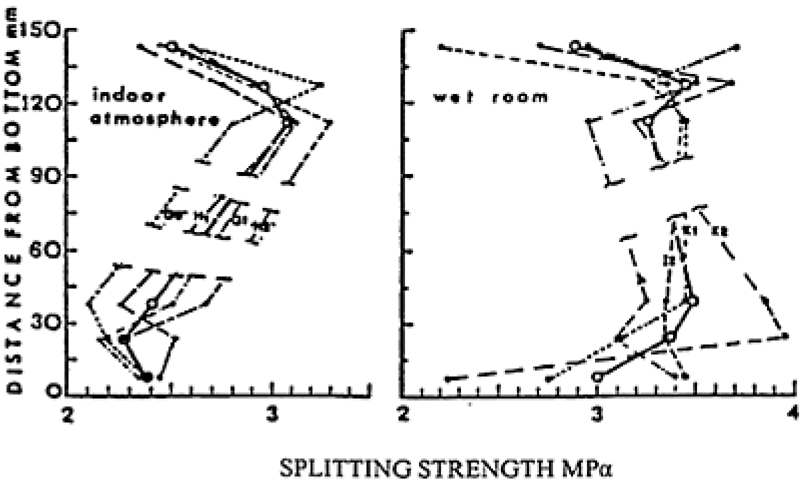

Figure 1 Tensile strength of concrete accross a 150 mm thick slab, depending on curing method (Tassios et al., 1991)

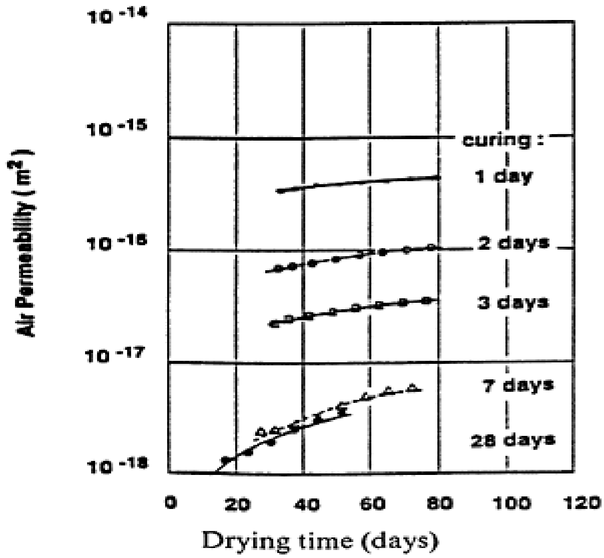

Figure 2 Effect of curing on air-permeability of cover concrete (Hong et al., 1989)

Moreover, the significance of curing on skincrete (the ten to thirty millimeters external layer of any concrete-element) cannot be overemphasised. For small height building elements, at the U.L.S., skincrete is the most important part of the compression zone; and its strength is highly dependent (Fig.1) on curing. More generally, however, this concrete layer (covering reinforcement and tendons) determines durabi lity in a decisive way: Permeation properties are highly dependent on curing (Fig.2). By way of consequence, designers have an interest to rely on good curing rather than on temporary coatings (or additional safety factors…).

In conclusion, against all the aforementioned undesirable structural effects, curing should be given an important place in design documents: Detailed and demanding specifications on curing (including specific quality control and separate unit prices) proved to be an efficient means to this end.

Falsework and Centring

Surprising as it may seem, temporary works of (literally) basic structural importance, are sometimes left out of the design documents; they are simply labeled as “construction topic”!

A steel or timber scaffold, however, is a structure par excellence, which may frequently need analytical verification of several requirements, such as (indicatively):

- Acceptable vertical deformation under dead load and compaction operations.
- Redistribution of dead load reactions during tensioning of tendons.
- Horizontal displacement under local deviation forces in curved pipes for concrete pumping.
- Adequate stiffness versus seismic aftershocks when concreting has to be continued after a main earthquake event.

Besides, striking without damage of the main structure, may occasionally be a difficult operation needing careful design of lowering devices. The failure of the first dome of Aghia Sophia (Fig.3) in Constantinopolis (563 A.D.) was partly attributed to the vibration produced when the 80 m high scaffold was stricken; the second time, however, a water layer brought in the Church on purpose, offered a complete damping to the enormous columns and beams falling down.

Detailing

Thanks to that part of experience (or chapter of Codes) called “Detailing”, a lot of verifications are not carried out by calculation; the relevant requirements are covered via practical rules. However, this simplification does not reduce the importance of the respective checks: Most of them are meant to insure safety, e.g.

- Of the steel bar itself: “Bending should be made with the aid of appropriate mandrels”, or, “Welding should not be made near bends of bars”.
- Of the anchorage and the durability of bars: “A specified minimum concrete—cover should be provided”.

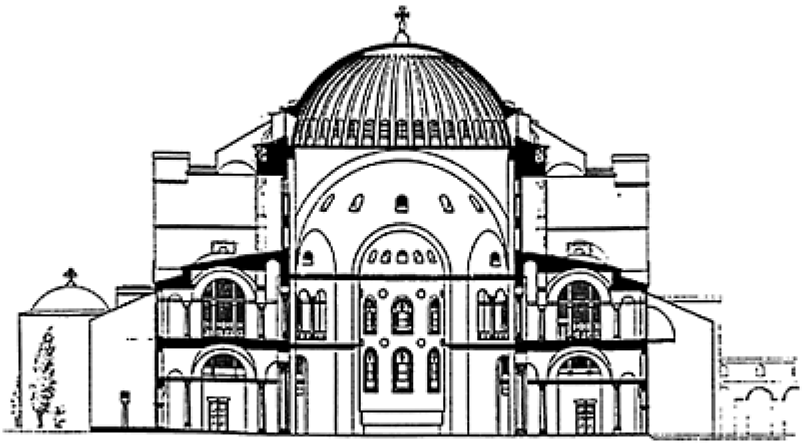

Figure 3 The pavement of the Aghia Sophia church (Constantinopolis, 563 A.D.) was water flooded in order to dampen the impact during striking of the dome after its reconstruction (Re. 4).

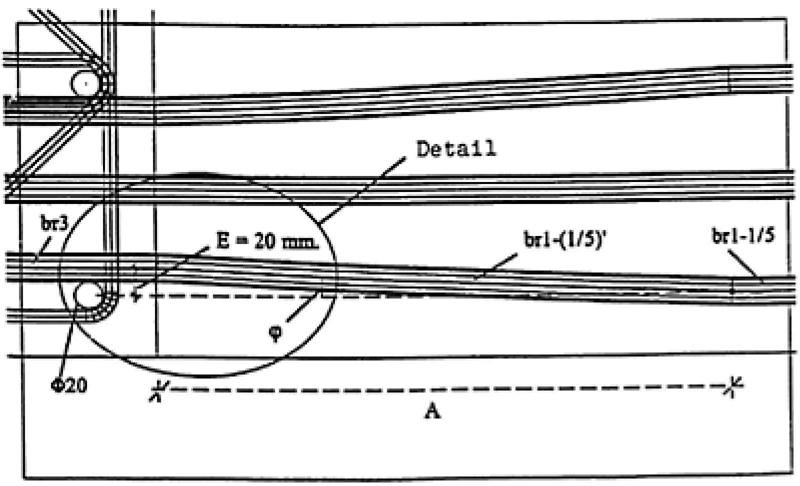

Figure 4 Necessary deviations of horizontal bars of a beam

Other such rules refer to serviceability, e.g.

• Of a slab: “Minimum top reinforcement against hygric or thermal cracking”.

Finally, several other detailing rules are intended to cover the inevitable uncertainties which otherwise would have undermined the applicability of calculation models (e.g. minimal thicknesses of concrete sections, maximum steel ratios, etc.).

This short reminding of the basic importance of this part of Design, may be useful per se; but it may also help to further harmonise design-versus-construction procedures: A designer deing conscious of the fundamental importance of these rules ("secondary" rules are called by some non-engineering minded people), will find ways to insure their correct application during the construction stage. To this end, these are at least two things to be reminded:

First, make your drawings as attractive and friendly to the Contractor as possible: the actual weaponry of C.A.D. is very helpful in this respect. Frequent "didactic" reminders illustrating important construction principles should be made on these drawings; do not feel you will offend any body if you repeat "well known" things, in a well illustrated way however. Second, establish a special category of checking of design documents, of buildability that is. The main problem in this respect is to identify areas:

- of "congested" reinforcement, requiring modifications in detailing, or an explicit warning about the way you envisage to overcome the local difficulty in concreting, and
- of "impossible" reinforcement, i.e. a geometry of steel bars which cannot be implemented because other bars occupy the same space; three dimensional coloured representations of such difficult areas (e.g. beam-column joints) may be used for such checkings, using special softwares (Fig.4) and (Fig.5) which may automatically make the necessary modifications of detailing.

CONSTRUCTION vs. DESIGN

One of the most controversial issues in Civil Engineering is the naive view that a designer in this field of industry "must" know everything in advance, to such an extent that a complete and precise budget and time schedule would be prepared and fully respected. Due, however, to the very nature of this peculiar "industry" (open air and nomadic factory, strong dependence on environmental data, no serial-production, non-certified personnel, etc.), such a "complete and precise" knowledge in advance, in most cases, is simply impossible: E.g. extensive soil investigation, adapting job site operations to medium range meteorological forecasts, exclusive use of certified personnel for every construction step, and the like, despite their socially unacceptable costs, would never suffice to overcome the inherent peculiarities of this industry.

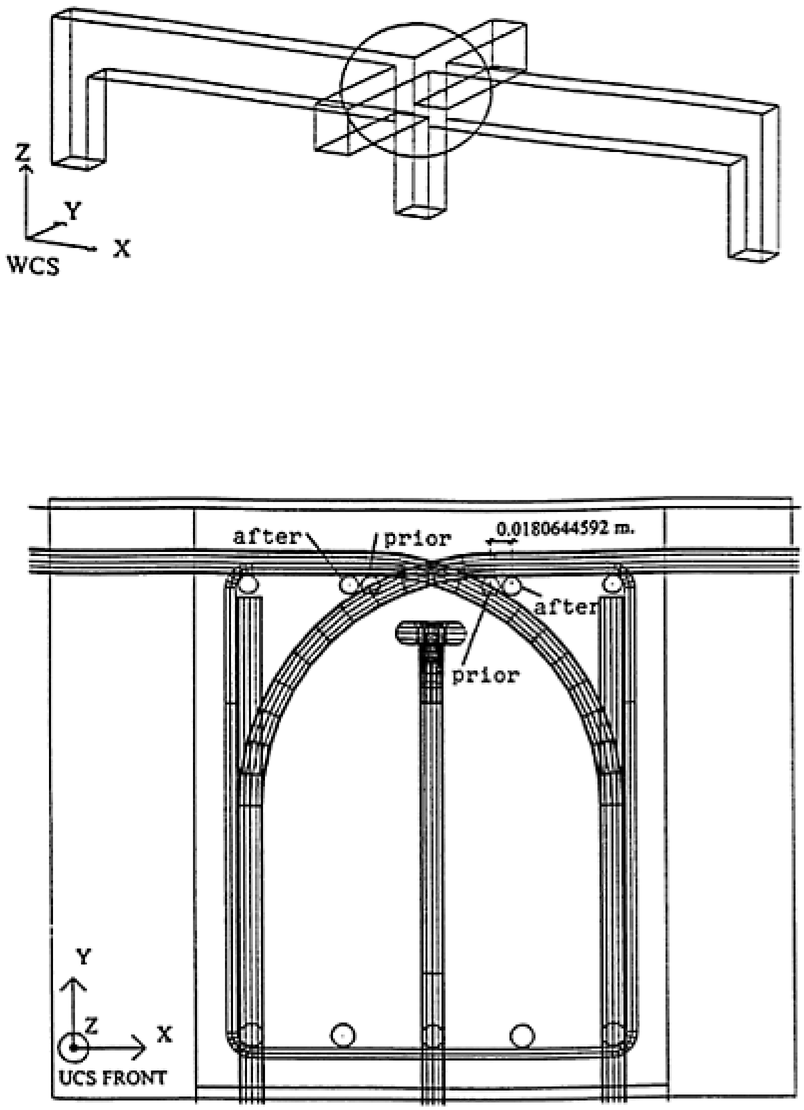

Figure 5 Modification in placing top bars of beam in a beam/column joint

Optimisation is reached by means of a more flexible scheme including a considerable stream of feeding-back information from Construction stage; not always though. It is therefore our professional duty to make it formal, instrumental and socially recognised. That is why, Civil Engineering Associations have to better specify and officialise this hybrid stage ("design in the making"), together with pertinent honoraries and clear responsibilities. This way, global responsibility will be re-established, and "interface leakage" will be minimised.

Regarding structural engineering, more specifically, it would suffice to mention here some of this feeding back information which should confirm or complete or modify "initial" design documents. Thus, (indicatively only), Contractor

- Controls the completeness of design documents (especially those related to soil conditions, temporary excavations and design of centring), and notifies the Owner accordingly.
- Informs Designer about construction documents prepared by site-engineers.
- Communicates site-records related to construction operations (e.g. pumping of ground water) which may affect redistribution of action-effects of already built parts or of neighboring buildings.
- Tendons' tensioning records and deviations from expected elongations should be counter-singed by the designer.

A similar flow of information should be initiated from the Supervising Engineer including records on operations, measures, tests and particular events.

Finally, it is interesting to notice a modern tendency towards a more closely related (almost amalgamated) design and construction stages. This is for example the case of in situ concreted segmental bridges, where maturity indices (of each segmental concrete) feed a special algorithm, defining the initial prestressing forces needed to keep the desired extrados line. Similarly, continuous monitoring of decisive structural estimators may mobilise appropriate strengthening interventions (pre-designed) in important monuments.

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THE DESIGN OF FRAMED STRUCTURES-A PROPOSAL FOR CHANGE

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ABSTRACT. The current approach to the analysis and design of reinforced concrete frames is considered starting from the assumption that adequate ductility can be provided. It is shown that the use of multiple arrangements of ultimate load is unnecessary and that the safety factors could be significantly reduced. This reduction is sufficient to make serviceability the critical design condition. An experimental programme designed to investigate the behaviour of frames at ultimate is briefly described. Preliminary results show that current analysis methods, which ignore membrane effects can lead to distributions of moment which are wildly different to what actually occurs. An alternative design approach, based on design primarily for serviceability is outlined.

Keywords: Reinforced Concrete Frames, Design, Analysis, Load arrangements, Safety factors, Membrane forces, Serviceability.

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INTRODUCTION

The approach almost universally used for the design of reinforced concrete framed structures was originally developed when design methods effectively assumed that reinforced concrete was an elastic-brittle material. As our understanding of reinforced concrete has developed, the design approach has been somewhat modified, but not really changed. Is this approach still the most appropriate in the modern age or would something different be more appropriate? This paper examines some aspects of this question and concludes that changes could, with advantage be made.

The approach originally developed for analysis and design can be summarized as follows:

(i) Elastic analyses are carried out for various arrangements of the loads to give a bending moment envelope. In most countries the arrangements used for obtaining the bending moment envelope were, and still are:

(a) alternate spans carrying the maximum load while the other spans carry the minimum load.
(b) each pair of adjacent spans considered in turn carrying the maximum load while the remainder carry their minimum load.

(ii) Critical sections are detailed to provide the required strength using permissible stresses obtained by dividing the specified material strengths by appropriate safety factors.

The realization that reinforced concrete was not elastic but had a degree of ductility led to the introduction of redistribution (initially commonly up to 15%). The development of the Limit State approach led to the use of design ultimate loads and material properties. These were obtained by applying partial safety factors to the loads and material strengths. Further research led to increases in the permitted amount of redistribution. A more recent change in BS8110 [1] in the UK has resulted in some simplification of the load arrangements by substituting a single load case of maximum load on all spans for the 'adjacent spans loaded' case.

It will be seen, however, that the changes have been incremental. The realization that reinforced concrete can be designed to be effectively plastic has not led to a full rethink of the logic of the design process.

Two elements of the design process will be considered in this paper: the employment of load patterns for the ultimate load analysis and the probabilistic consequences of continuity. In both cases it will be suggested that current methods are unnecessarily conservative provided adequate ductility it available. A further matter that will be discussed is the relevance of current methods of frame analysis to the actual behaviour of frames at ultimate. It will be suggested that th the relevance is limited as inelastic behaviour leads to major differences from current analyses.

LOAD ARRANGEMENTS FOR ANALYSIS FOR THE ULTIMATE LIMIT STATE

Assuming that all sections of a beam have adequate ductility, failure of a reinforced concrete beam will be the result of a mechanism forming. This mechanism will consist of elements of beam connected by plastic hinges. When a beam is designed, reinforcement is provided to match as nearly as is practical the strength required at the critical sections. Usually the sections where the design strength most closely matches the required strength will be over the supports and near mid-span but, where reinforcement is curtailed, there may also be other points where the designed strength approaches the required strength. At all other points the beam will have a bending capacity greater than required by the bending moment envelope. This limits the positions where hinges are likely to form and hence limits the possible failure mechanisms that need consideration.

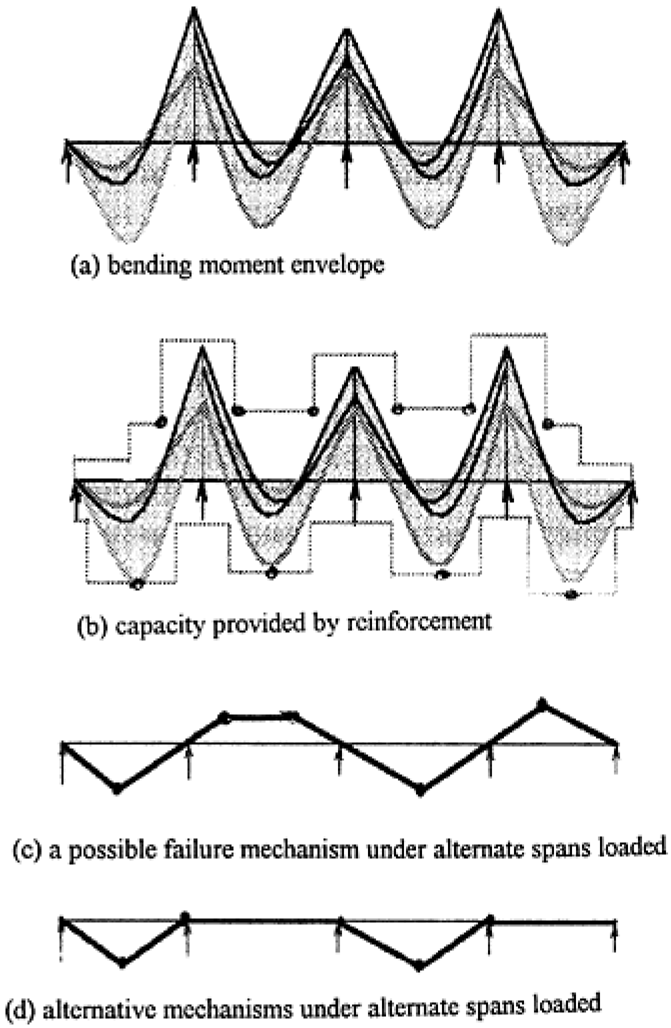

Figure 1. Analysis, design and failure of a continuous beam.

When a continuous beam is analysed using the classical load arrangements set out above, the bending moment diagram for each load arrangement is calculated and the results superimposed to give a bending moment envelope. Such an envelope is shown schematically in Figure 1(a). Figure 1(b) shows the moment capacity likely to be provided by the reinforcement and the possible locations for hinges. It will be found that two mechanisms are practically feasible: a mechanism with sagging and hogging hinges in alternate spans and a mechanism having a hogging hinge at two adjacent supports and a sagging hinge near mid-span. These are shown in Figures 1(c) and 1(d). It can easily be shown [2] that the critical mechanism is that shown in Figure 1(c) where all hinges form in regions of the beam where the design moment is defined by the 'alternate spans loaded' load arrangement. The support moment, which is defined in the analysis by the 'adjacent spans loaded' arrangement, would have to be reduced to that required for the 'alternate spans loaded' case before the mechanism shown in Figure 1(d) became critical. Thus, provided a ductile failure can be guaranteed, it should only be necessary to carry out analyses for the 'alternate spans loaded' arrangement. Further simplification can be achieved by considering the practicality of the mechanism shown in Figure 1(c). In monolithic framed structures, the supports do not permit free rotation but are monolithic connections with columns and the failure mechanism can only develop if the beam-column joint can rotate sufficiently. Furthermore, current detailing rules mean that the top reinforcement is not curtailed till well beyond where it is needed and hence a hinge at the curtailment point of the top steel will have a strength well in excess of that required by the bending moment diagram. It thus seems that a mechanism requiring hinges in more than one span is most unlikely to occur. If so then the only practically likely failure mechanism is the three hinge mechanism shown in Figure 1(d). To ensure that this mechanism has adequate strength it is only necessary to ensure that, for beams supporting uniformly distributed loads, the support and span moments satisfy the relation:

$$\frac{w_u l^2}{8} = \frac{M_{u.span}}{4}\left(\frac{j+1}{\alpha} + \frac{i+1}{1-\alpha}\right) \qquad [1]$$

$$where\, \alpha = \beta\left(-1 + \sqrt{1+\frac{1}{\beta}}\right) \quad and \quad \beta = \frac{1+i}{i-j}$$

i and j are respectively the ratios of the left-hand and right-hand support ultimate moments to the ultimate span moment, $M_{u.span}$.

This can conveniently be achieved by the simple expedient of analysing the beam for the single load arrangement of maximum design load on all spans.

Thus, provided adequate ductility can be ensured, design for ultimate in flexure does not require analysis for multiple load arrangements and the construction of a bending moment envelope. The effect of this is a significant reduction in reinforcement quantities.

PROBABILISTIC CONSIDERATIONS

It has been argued above that the only failure mechanisms which needs to be considered are those occurring within one span (the three hinged failure). A statically determinate structure only needs to reach its ultimate capacity at one section to fail. From a probabilistic point of view, it will be seen that a continuous structure will have a lower probability of failure than a simply supported beam because the probability of three hinges being simultaneously understrength must be smaller than the probability of a single understrength section. If, therefore, the level of safety currently required for simply supported beams is considered adequate, then the safety of continuous structures must currently be excessive and a lower safety factor should be admissible. A simplified analysis can be carried out to illustrate this.

For a continuous beam with equal support moments to carry the same load as a simply supported beam with the same dimensions,

$$M_{u.ss}=M_{u.sup}+M_{u.span}=w_u l^2/8 \qquad [2]$$

where $M_{u.ss}$= the ultimate moment capacity of the mid-span section of the simply supported beam.

$M_{u.sup}$= the ultimate moment capacity of the supports of the continuous beam

If the coefficient of variation of the ultimate moment is v and is assumed to be independent of the reinforcement ratio, then the standard deviation of the ultimate load of the simply supported beam is given by:

$$\sigma_{w.ss}=8vM_{u.ss}/l^2 \qquad [3a]$$

and that for the continuous beam by:

$$\sigma_{wc}=8v\sqrt{(0.5M_{u.sup}{}^2+M_{uspan}{}^2)}/l^2 \qquad [3b]$$

For typical values of the ratio of support to span moment, it can easily be shown from Equations 2 and 3 that the ratio of the standard deviation of the capacity of the continuous beam to that of the simply supported beam is about 0.6.

Assuming that the distributions of strength are normal then, for a constant risk of failure, Eurocode 1 Part 1 [3] states that the partial factor may be chosen such that a constant value is obtained for the safety index β where:

$$\beta=(w_{um.}-w_{ud})/\sigma_w$$

in which

w_{um} = mean ultimate load capacity

w_{ud} = design ultimate load capacity

σ_w = standard deviation of ultimate strength.

Using the normal definitions for characteristic and design strengths and assuming that the load capacity is proportional to the material strength, it can be shown that the required partial safety factor for a continuous beam is given by:

$$\gamma sc=\gamma ss \quad \frac{(1-0.98v)(1-\beta v)}{(1-1.64v)(1-0.6\beta v)}$$

γsc = partial safety factor for continuous beam

γss = partial safety factor for a simply supported beam

Insertion of reasonable figures into this formula suggest that a reduction in safety factors of up to about 15% might be reasonable.

A more detailed study of this issue was carried out by Beeby for the Reinforced Concrete Council (unpublished). This took account of the variabilities of steel and concrete and of the section geometry. The effects of correlations between the properties at the three critical sections was also considered. This suggested that, if correlations between the three critical sections are ignored then a reduction in safety factors of about 15% would be reasonable. Taking into account the maximum likely degree of correlation leads to roughly a halving of this reduction. Combined with the reductions in steel area resulting from discarding multiple load arrangements, an overall saving in flexural reinforcement of up to 30% seemed possible without reducing the safety below that accepted for determinate structures.

OTHER FACTORS

There are a number of other issues being discussed nationally and internationally which, if accepted would lead to a further reduction in the quantity of reinforcement used in frames. The following lists the main areas of discussion.

(i) reduction in the partial safety factor on reinforcement from 1.15 to 1.05. The British Standards Institute (BSI) has recently published a draft amendment to BS8110 proposing this. Its effect would be to reduce the areas of main flexural steel by around 8%.

(ii) reduction of the partial safety factor on dead loads. There is considerable discussion within the Eurocode drafting committees of the possibility of reducing the partial factor on dead load from 1.35 to possibly 1.15. The argument for this is that the dead load can be assessed with greater accuracy than is indicated by a factor of 1.35. The pressure for a lowering of safety factors in the Eurocodes arises from interaction with the old Eastern Bloc countries which use, and have used for many years, considerably lower overall levels of safety than those used in the West. They claim that these lower safety levels have not caused any problems.

(iii) increase in the specified characteristic strength of reinforcement from 460 N/mm^2 to 500 N/mm^2. The European prestandard for reinforcement [ENV 10080] only recognises 500 grade reinforcement and it seems likely that this will be accepted. This does not directly affect safety but does have an effect on the stress in the reinforcement under serviceability loads.

DISCUSSION

The acceptance of any, or all the proposals discussed above lead to a reduction in the areas of flexural steel employed in reinforced concrete frames. This, in turn, leads to an increase in the stress levels in the reinforcement in service which leads to an increase in service deflections and crack widths. Also critical to the proper performance of frames in service is the requirement that the reinforcement shall behave elastically under service conditions. Any inelastic behaviour will lead to the development of large permanently open cracks which will generally be unacceptable. Both BS8110 [1] and Eurocode 2 [4] limit the stress in the reinforcement under service load to $0.8f_y$ to avoid this possibility. At present it is not generally necessary to check for this since the safety factors under ultimate conditions and the limits on redistribution generally ensure that the limit is not exceeded. Unfortunately, this is not the case if the safety factors are reduced to the extent which seems possible for framed structures and the check on the service stress becomes the controlling factor in the design, (i.e. the area of flexural steel is governed by the limitation of the stress under service load to $0.8f_y$ and not by the ultimate flexural strength.)

MEMBRANE EFFECTS IN CONCRETE FRAMES

It has been recognised for very many years that, if an under-reinforced member is restrained against lateral expansion, its strength can be substantially enhanced due to the development of in-plane forces which effectively prestress the member [eg 5]. The forces develop because when a reinforced beam cracks and, more especially, when yield develops at any section, the beam, if unrestrained, will expand. If this expansion is inhibited by restraint at the supports then an in-plane force will develop which will be a function of the difference between the expansion which would occur in an unrestrained beam and that which occurs in the restrained beam. In slabs where restraint to expansion may be provided either by other surrounding slab panels or by stiff elements in the structure such as core walls, the increase in strength resulting from restraint can be very large and has been the subject of extensive research. It has not generally been incorporated into design codes because of the difficulties in defining the restraint conditions, the enhancement in strength of beams in framed structures could be expected to be generally small as restraint is commonly only provided by fairly flexible columns. The possibility of using this strength enhancement in design has never seriously been considered. What does not appear to have been recognised is that, regardless of whether account is taken of the possible increase in capacity of the beams or not, if lateral forces develop then they have to be supported by the columns. The columns must also be capable of sustaining the deflections resulting from expansion of the beams. An extensive research programme is currently in progress in the Civil Engineering Department at Leeds on the actual ultimate behaviour of framed structures. The results so-far from this project illustrate very clearly that the actual conditions under ultimate conditions in a frame are very different from those which current analysis methods would predict. An outline of the test programme and a summary of the results so-far are given below.

EXPERIMENTAL PROGRAMME

The test specimens aim to model conditions in the end columns and end span of a rectangular reinforced concrete frame (see Figure 2). The specimens are designed to be roughly half scale models of a practical frame. The overall height of the column between points of contraflexure was chosen as 3.6 m and the length of the beam (corresponding to approximately 2/3 of the span) was 2.2 m. It is believed that, at this scale, size effects should not be significant. The test rig is shown in Figure 3. The objective is to be able to load the beam and also provide an axial load in the column while permitting any axial load in the beam which develops to be measured. Considerable difficulties were encountered in arriving at a set up which would permit reliable assessment of the axial loads but this was eventually achieved.

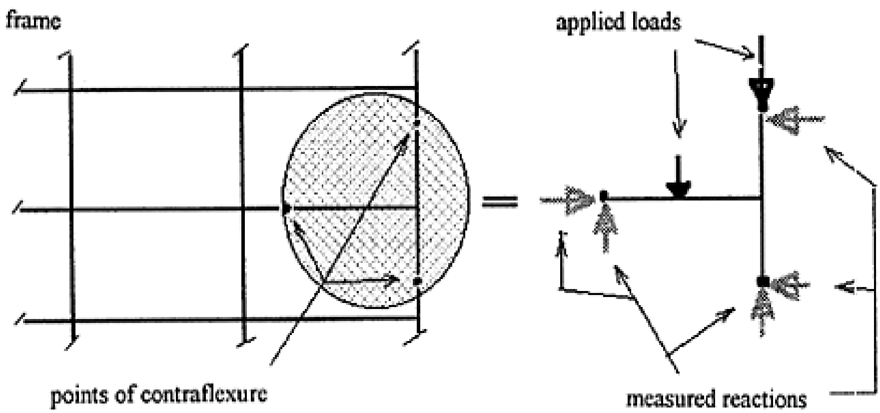

Figure 2. Concept of experimental set-up

Results will be presented for a series of four tests where the beam and column breadth was 150mm and the overall depth of both members was 250mm. The axial load in the column and the reinforcement ratio in the beam were the principal variables investigated. Table 1 shows the principal results.

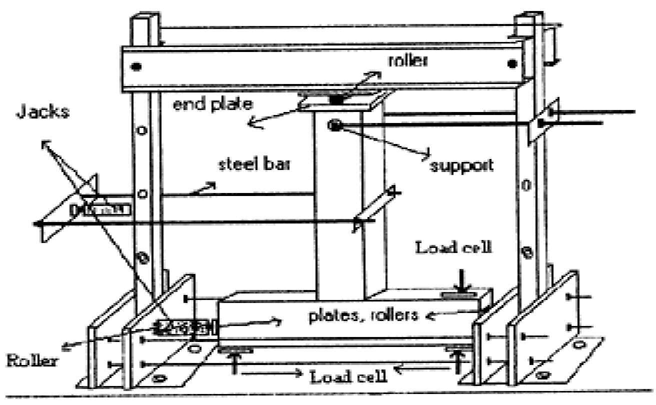

Figure 3. Test arrangement.

Table 1. Details of tests.

Test No.	Column load (kN)	support steel %	span steel %	Design ultimate load (kN)	Test Ultimate load (kN)	Test Ult Membrane force (kN)
1	200	0.6	0.6	73	122	76
2	200	0.9	0.4	84	115	51
3	200	1.6	1.6	190	200	42
4	420	0.6	0.6	73	106	73

It will be seen that the capacity of the beams have been increased somewhat, with greater increases with lower reinforcement ratios, and that significant membrane forces have developed. The forced developed have been between 1 and 2 N/mm^2 at ultimate. The most significant effect on behaviour is, however, the bending moments imposed on the columns. This is illustrated in Figure 4 which compares the bending moment diagrams obtained from analyses ignoring the membrane forces with those which were measured at maximum load on the beam.

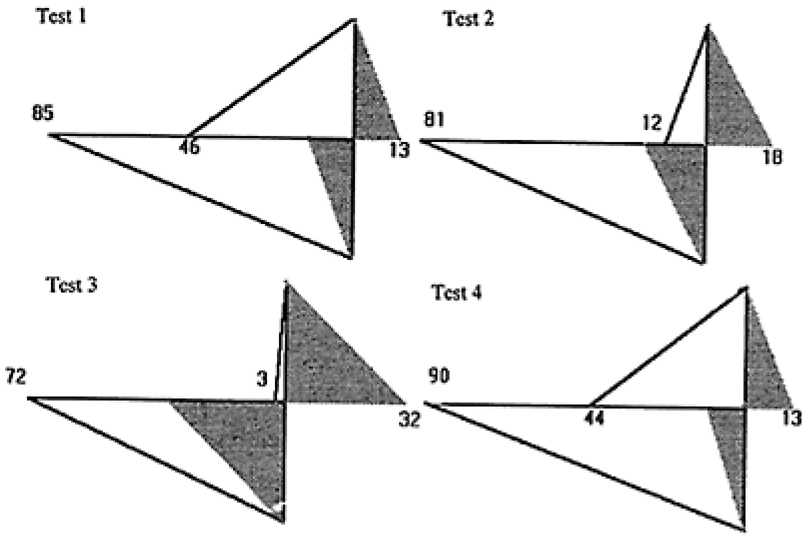

Figure 4. Experimental ultimate bending moment diagrams in columns (the shaded areas indicate the calculated ultimate bending moments ignoring membrane effects).

It will be seen that the actual column moments may be up to seven times the calculated values if membrane forces are ignored. Specimens 1 to 3, which had low axial loads on the column, all failed finally by failure of the beam. Specimen 4, and other tests where the higher column load has been applied, failed in the column. Thus, not only does the normal method of analysis not predict the moments which occur in the frame near failure, it may also not correctly predict the mode of failure.

Other Investigations, such as the tests by Lahlouh and Waldron [6] show similar results.

It has been found that more rigorous, non-linear analyses using finite elements (ABAQUS) can predict the behaviour of the frames with reasonable accuracy provided allowance is introduced into the analysis for the development of bond failure in the hinge regions.

CONCLUDING DISCUSSION

It has been argued that there are a number of changes that could logically be made to the design of reinforced concrete framed structures for flexure which could result in significant economies and that there are other developments nationally and internationally which could lead to further reductions in the areas of flexural reinforcement. The study carried out for the Reinforced Concrete Council shows that any significant reduction in the area of flexural reinforcement will lead to the requirement that there is no yield of the reinforcement under service conditions being the critical design condition rather than ultimate strength. The research on membrane effects in frames shows that the normal type of analysis used for framed structures, which ignores membrane effects, leads to some conservatism in the estimation of the strength of the beams but a major underestimation of the moments which may be imposed on the columns. This underestimate is sufficiently large that, while the design method assumes that failure will occur in a ductile manner in the beams, there are likely to be cases where the actual failure will be a brittle failure of the column. The present method of analysis is thus misleading at the ultimate limit state.

It seems arguable, therefore, that the current analysis for the ultimate limit state is not particularly helpful and that there might be advantages in carrying out design for the serviceability limit state instead. The proposal is the following for beams:

(a) carry out the analysis of the structure under the service loads using the classical load arrangements of 'alternate spans loaded' and 'adjacent spans loaded.

(b) calculate the required reinforcement using an elastic, modular ratio method with a limit of $0.8f_y$ on the stress in the reinforcement.

(c) check cracking, deflections and any other relevant service conditions using the service steel stress.

(d) check shear. In principle, this check should be done under ultimate load conditions but it is likely to be easier to recast the shear check in service load terms.

The advantages in this approach are:

(a) a significant saving in flexural reinforcement quantities. The Reinforced Concrete Campaign study suggests that this may be about 25%.

(b) by concentrating on serviceability, design attention is concentrated on aspects of performance which actually concern the client and the user such as excessive deflection and which can be measured rather than the almost entirely hypothetical conditions that might occur near failure.

(c) use of an elastic analysis will ensure that the requirements for ductility at critical sections at ultimate is a minimum and hence no special measures beyond some overall limit on neutral axis depth will be necessary to ensure that adequate ductility is provided.

The disadvantage, which may be fatal to the suggestion, is that it will require a major revision to current design procedures. This will be resisted fiercely by designers.

CONCLUSIONS

(1) From studies of the behaviour of beams it is concluded that, at ultimate, there is no justification for the consideration of the current load arrangements in the analysis.

(2) Consideration of the probability of failure of continuous beams relative to that of determinate members show that the safety factors for continuous members could be significantly reduced.

(3) If (1) and (2) above were implemented then serviceability considerations would be found to control the quantity of flexural reinforcement rather than ultimate.

(4) Consideration is currently being given to reducing the partial safety factor on reinforcement strength, reducing the partial safety factor on dead loads and increasing the specified strength of reinforcement. Each of these, if adopted would further increase the criticality of serviceability conditions and reduce the criticality of strength considerations.

(5) Research shows that the current methods of analysis for the ultimate limit state, which ignore the influence of membrane forces, can provide very misleading predictions of behaviour.

(6) Actual problems with structural behaviour are practically always serviceability problems rather than safety.

(7) Taking all the above into account suggests that design might more sensibly be carried out for the service condition rather than ultimate.

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DEVELOPMENT OF MOMENT CONTINUITY CONNECTION FOR SIMPLY SUPPORTED PRECAST CONCRETE BRIDGES

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ABSTRACT. Current methods of making simply supported precast prestressed bridge girders continuous are generally limited to new construction and can not be used to convert existing simply supported span bridges into continuous or integral bridges. To address this limitation and to minimise the cost of bridge maintenance, a study on methods of continuity has been carried out. The proposed connection provides for both hogging and sagging moments over the support by introducing localised trapezoidally shaped insitu concrete infill between the precast beams with continuity reinforcement placed at deck and soffit levels. The paper reviews the various methods currently available and makes comparisons with the proposed approach. A series of experiments which were designed to test the effectiveness of such connections are described.

Keywords: Prestressed concrete, Bridge beams, Simply supported, Continuous integral bridges, Construction, Maintenance.

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INTRODUCTION

There are many advantages in making short or intermediate span bridge decks simply supported during the construction phase. These include rapidity of construction and economy. However it is increasingly recognised that maintenance problems are associated with extensive below-deck deterioration due to ingress of drainage water through expansion joints, particularly when laden with de-icing salt. Additionally smooth jointless construction improves vehicular riding quality and diminishes vehicular impact stress levels. There is therefore considerable interest in reducing the number of expensive expansion joints, producing continuous deck construction or integral bridges. The basic building block is still the pre-tensioned beam, and consideration has been given to how continuity can best be accommodated practically, to ensure that both the hogging and sagging moments can be adequately designed for the limit states of strength and serviceability. There is interest in maintaining a competitive market edge for future bridges, as well as the upgrading of existing bridges. Sagging moments at the supports may be due to loading, settlement, creep, shrinkage, and temperature effects. The paper reviews existing methods, indicating advantages and disadvantages, and proposes an alternative method which is appropriate for both old and new structures. In particular the paper describes preliminary tests carried out to establish sagging moment capacity and the moment transfer mechanism for such connections.

EXISING METHODS OF PROVIDING CONTINUITY CONNECTION [1, 2, 3]

The following Types of construction shown in Figure 1 have already been used:

1. Wide Insitu Integral Crosshead.
2. Narrow Insitu Integral Crosshead.
3. Integral Crosshead Cast in Two Stages.
4. Continuous Separated Deck Slab
5. Tied Deck Slab
6. Prestressed Deck Continuous Over the Pier

Limitations/Problems

Method	Limitations/Problems
Type 1	1. Requirement of temporary supports.
	2. Projection of bottom continuity reinforcement beyond the ends of precast beams resulting in
	a. Congestion of reinforcement at the beam ends
	b. Reduces the benefit of standardisation
	c. Health and Safety hazards during transportation and erection
	d. Difficulty in launching of beams

Type 2	1. Spaces between beam ends are very narrow which cause difficulty in placing high quality concrete. 2. Projection of bottom continuity reinforcement at the ends of precast beams causes: a. —d. as for Type 1.

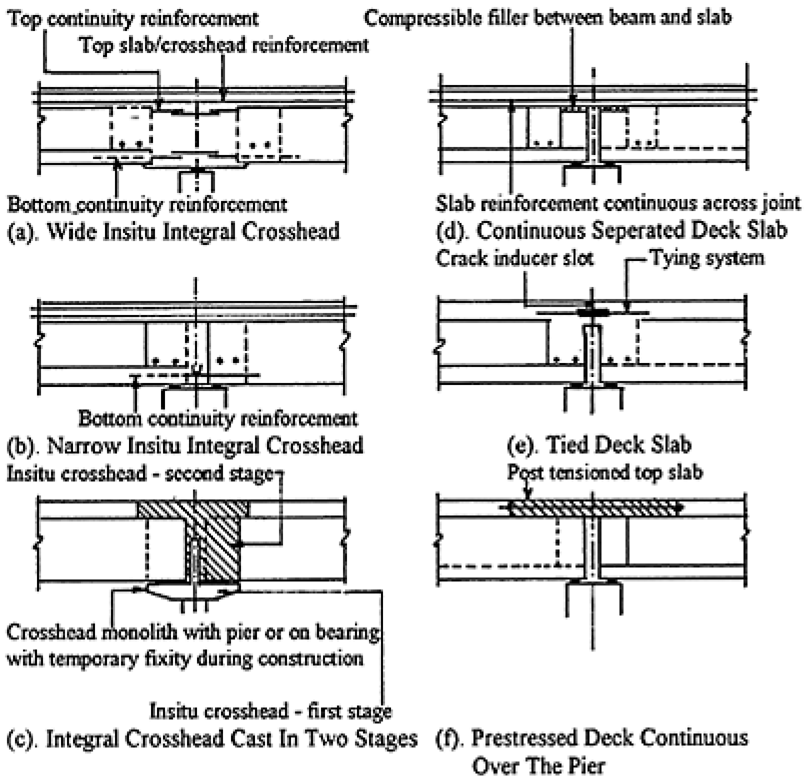


Figure 1 Methods of Continuity Connections (After Pritchard, et al.[1, 2, 3])

Type 3	1. No reduction of mid-span sagging moments due to live load, therefore no economic gain in terms of the size of beams used. 2. Proper procedure of alternate placing of beams on the pier has to be carried out to balance the load on the pier.
Type 4 and 5	As Type 3.1.
Type 6	No record reporting the adoption of this method.

Retrofit Bridges

None of the existing methods are suitable to convert existing simply supported span bridges into continuous/integral bridges.

PROPOSED CONNECTION—TRAPEZOIDAL INSITU R.C. DIAPHRAGM

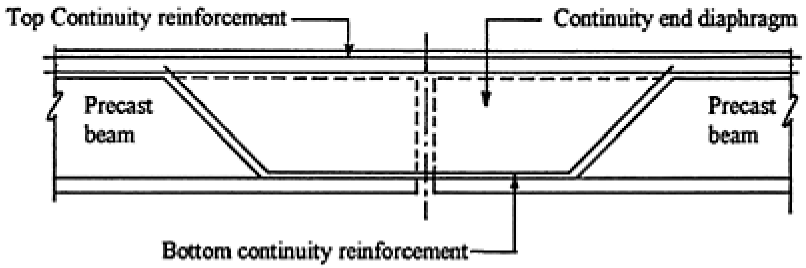


Figure 2 The Proposed Trapezoidal Insitu R.C. Diaphragm Connection

Description:

• Continuity over the pier is confined to the diaphragms and deck slab.
• Negative moment: reinforced concrete deck slab and precast beam.
• Positive moment: reinforced insitu concrete diaphragm.

Differences with existing methods. The connection is similar to Type 2 except that:

• There is no bottom continuity reinforcement at the ends of precast beams.
• The shape of the insitu reinforced concrete diaphragm is trapezoidal so the change of curvature between the two different cross-sections is smoothed.
• The reinforced concrete diaphragms are designed for positive moment.

Advantages:

• No continuity reinforcement at the ends of the precast beams—problem of congestion, health and safety hazards avoided.
• No temporary work is required.
• Minimal additional formwork and ease of construction.
• Can be adopted for converting existing bridges.
• Suitable for most bridges consisting of beams with bottom flanges side-by-side such a T-beams, inverted T-beams and M-beams.

TEST SPECIMENS, PREPARATION AND TESTING

Introduction

It is quite difficult to simulate a rigorous test for sagging moment resistance at a support in a continuous beam, particularly if primarily caused by temperature gradients or shrinkage. However the essential requirement is the ability to transfer the beam span moments through the interfaces of the insitu support connection satisfactorily. Since the real life situation could not be completely idealised it was decided to make initial comparisons on the basis of simply supported composite beams under four point sagging moment—Figure 3.

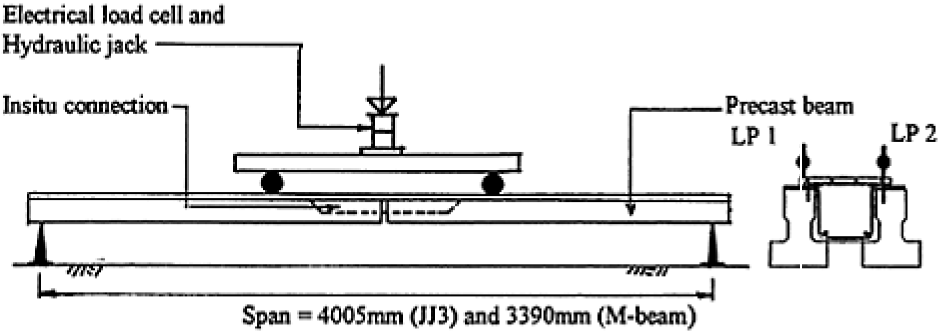


Figure 3 Four Point Loading Arrangement

Initial Model Tests

In an attempt to find a ready and cheap supply of model beams, Marshall JJ3 Flooring units [4] were initially used. However in the absence of shear reinforcement it was necessary to form composite insitu slab connections at the top interface by drilling and connecting shear connectors using epoxy resin. The composite beam was then formed by casting an insitu slab and central diaphragm as shown in Figure 4. Failure however occurred when large single horizontal web cracks forming at the ends of the precast beams at midspan. The cube strength of insitu concrete and precast beams during testing were 51 and 60 N/mm^2. Three Tests 1.1–1.3 were carried out on identical specimens, except that in Test 1.2 and 1.3 the precast beams and insitu slab were clamped between flanges—Figure 5, to prevent the development of these major splitting cracks. For Test 1.2 specimen, both ends of precast beams at the mid-span were clamped. For Test 1.3, only one side of the connection where failure occurred was clamped. The results are discussed later.

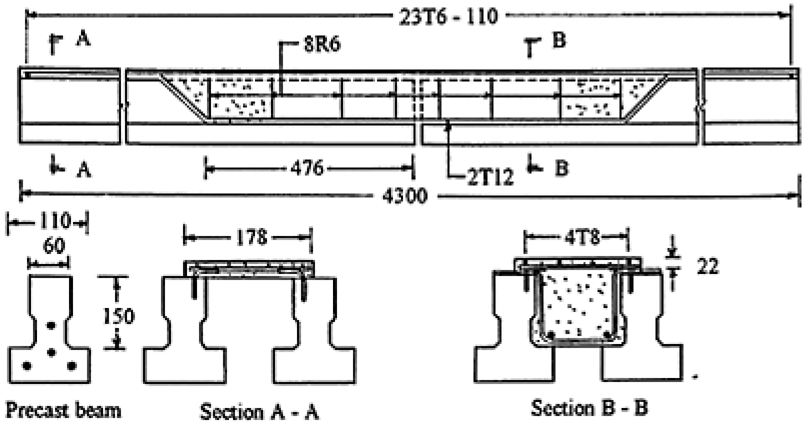


Figure 4 Details of the JJ3 precast beam and the connection—Initial Model

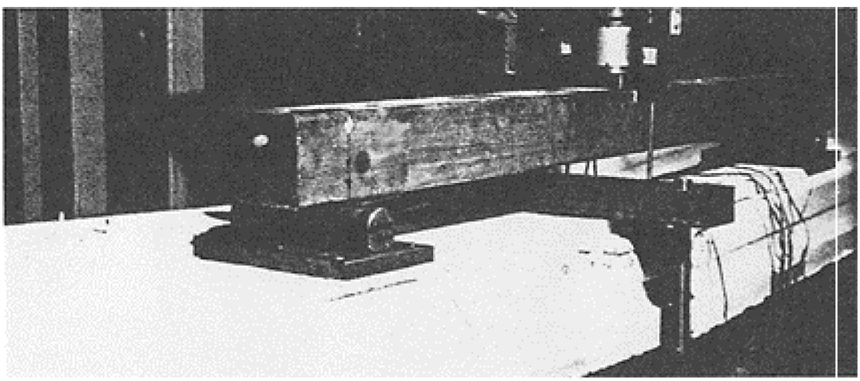


Figure 5 Vertical clamping arrangement for testing Model 1—Test 1.3

M-beam Model Tests.

Due to the major splitting which occured in the JJ3 beams tests, it was considered necessary to ensure that shear stirrups were provided at the ends of the precast beams. Composite prestressed beams were manufactured in the laboratory to the details shown in Figure 6. The cross section of the beam resembles the 'M-beam', and provides a realistic shape to the interface. The end stirrups were strain gauged and a plot of the strains was recorded from transfer stage to failure. The cube strengths of diaphragm and deck slab concrete and beams were 58 N/mm^2 and from 44 to 54 N/mm^2, respectively.

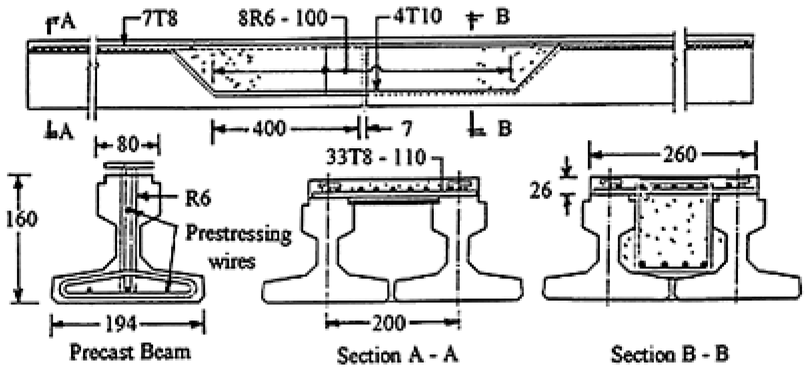


Figure 6 Details of the precast M-Beam Model, showing the connection

Test Procedure

The beams were supported temporarily at the mid-span position and end until the in-situ infill concrete had hardened. The general test arrangement for Initial and M-beam Model Tests is as shown in Figure 3. Two linear potentiometers (LP1 and LP2) were placed at either side of the mid-span position. Loads were gradually applied up to failure. During testing vertical deflections at the midspan and crack opening at the bottom of diaphragm were measured and recorded.

TEST RESULTS

Initial Model Tests

Figure 7 shows the central deflection plot of the composite beam against applied moment (with self weight) for Tests 1.1–1.3 while Figure 8 shows the cracking pattern for Test 1.1.

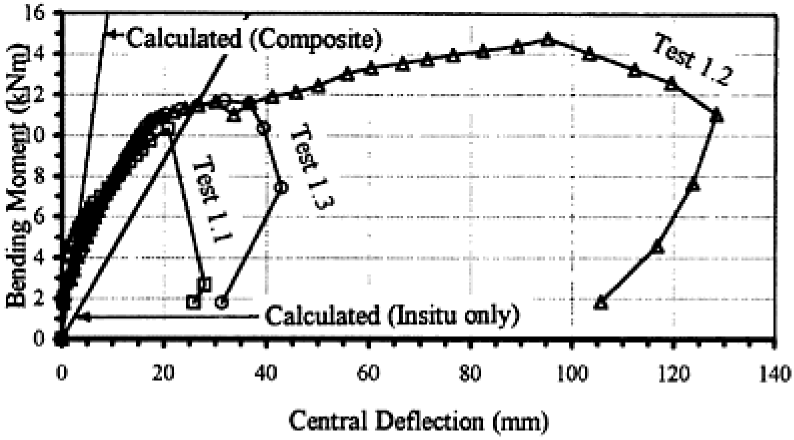


Figure 7 Bending Moment versus central deflections: Model 1—Test 1.1 to 1.3

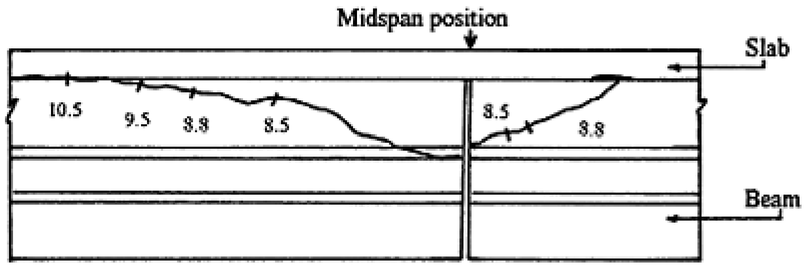


Figure 8 Crack pattern at failure: Model 1 Test 1.1

Unlike the other two tests, Test 1.1 behaved non-linearly up to a point approximately half of the ultimate failure moment. The failure was sudden at a bending moment of 10.5 kNm. In Test 1–2, linear behaviour was observed up to almost three quarters of the failure moment, where following extensive cracking the flexural stiffness had fallen one tenth of the initial stiffness. The failure moment was 15.1 kNm, i.e 44% greater than Test 1.1. Test 1.3 (which is practically a continuition of Test 1.1) behaved in a similar manner to Test 1.2 except that failure moment occured at 12.0 kNm, i.e only 14% greater than Test 1.1.

Figure 7 also shows the calculated mid-span elastic deflection for Tests 1.1 to 1.3. The results for Test 1.2 and 1.3 were broadly similar. The theoretical applied-moment vs central deflection data for composite beam-slab section agrees almost exactly with test results up to a moment of 5.9 kNm. Also, the final secant stiffness in the test agrees with the calculated stiffness based on the infill cross-section.

M-beam Model Test

Figure 9(a) shows the central deflection plot against applied moment. Non linearity set in at about 50 % of the ultimate failure moment. The failure was sudden and appeared to be due to flexural cracking of the bottom flanges of the precast beams at the ends of insitu infill concrete, and occurred at the applied moment of 12.0 kNm. Figure 9(b) shows the strains in the bottom longitudinal reinforcement of the infill concrete.

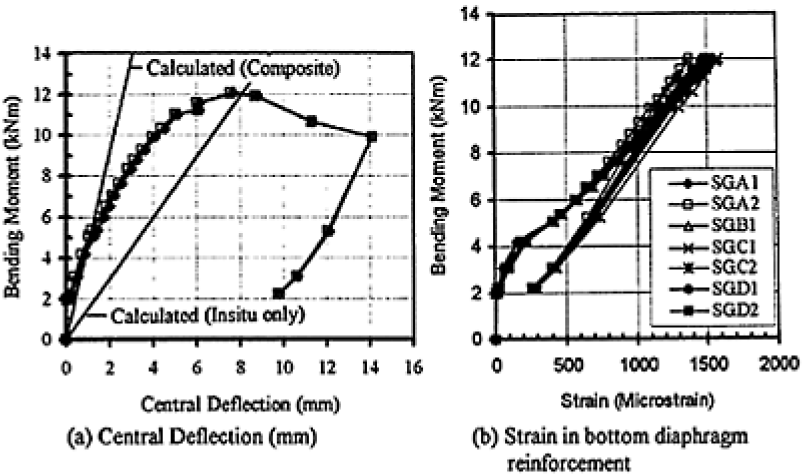


Figure 9 Variation of central deflection and diaphragm reinforcement strain in M-beam model test

Cracks appeared at a total bending moment of 9.6 kNm and started from the soffit of the bottom flanges of the precast beams near the ends of the solid infill diaphragm and propogated upwards as the bending moments increased as shown in Figure 10. No cracks appeared elsewhere in the connection. Strain distribution in the links of the precast beams at various distances from the connection are shown in Figure 10(a).

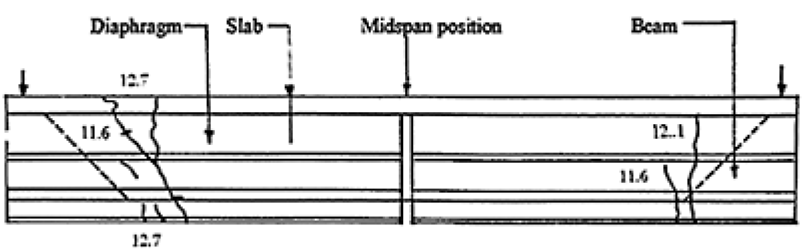
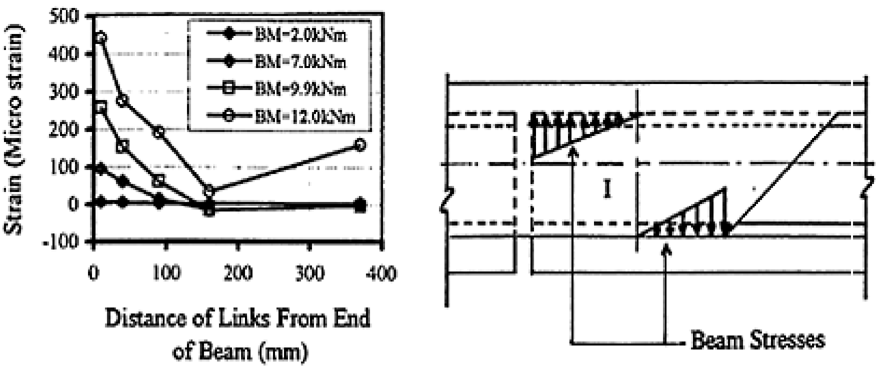


Figure 10 Crack pattern at failure: M-beam model test



(a) Distribution of strain in links (b) Possible moment transfer mechanism

Figure 11 Variation of link strain and possible moment transfer mechanism: M-beam model

DISCUSSION

Sagging Moments at Continuous Supports

The creep strains and thermal deformations which increase upward camber in prestressed concrete members also induce sagging moments at the supports in continuous construction. Flexural resistance must not only be active within the diaphragm where the flexural reinforcement is placed, but also beyond its end into the precast beam.

Initial Model Tests

In Test 1.1 cracks which initiated at 8.5 kNm, cut through the web of the beams on either side of the mid-span joint, and propagated towards the loading point (Figure 8). In Test 1.2 cracks initiated lower down in the beam at 6.5 kNm and followed a similar trajectory as above. Delamination along the slab-beam interface at 16.3 kNm instigated ultimate failure. The theoretical ultimate moment of resistance (using unfactored material data and rectangular compressive stress block method) was 11.0 kNm. Thus the ratio of test-to-calculated moment of resistance in the three tests are 0.95, 1.37 and 1.09.

M-beam Model Test

Figure 9(b) shows the strains developed in the bottom reinforcement of the infill diaphragm. The strain of 1500 microstrain (about three quarters of yield tensile strain) developed corresponding to the maximum bending moment of 12.0 kNm. It is therefore apparent that the continuity connection made up of insitu reinforced concrete diaphragm is capable of transferring the sagging moment from one span to another. Initial considerations indicate that moment continuity is produced by the direct bearing action of the insitu diaphragm against the precast beam flanges—the resisting couple acting about the centre of rotation I, as shown in Figure 11(b). Figure 11 (a) shows the strain distribution in the links of the precast beams adjacent to the diaphragm. As the applied bending moment increased, the tensile strains in the links gradually increased and the centre of rotation of the couples created in the connection moved away from the edge of the beam toward the ends of the solid infill diaphragm. At the failure bending moment, tensile forces in the links on one side of centre of rotation were balanced by a point compressive force exerted by the infill on to the bottom flange of the beam which caused cracking in the precast beamsshown in Figure 10. This prevented the development of the theoretical ultimate moment of resistance of the infill of 19.51 kNm. The ratio of test/calculated bending moment was 0.62.

CONCLUSIONS

The ultimate sagging moment capacity of the connection formed of JJ3 model precast beams, i.e. the initial tests, but excluding shear reinforcement was higher than its ultimate value. The mode of failure, however, involved a single large web crack and an unacceptably sudden failure, indicating a general weakness of the unreinforced web zone adjacent to the joint. It was decided that it would not be possible to develop the research using these units. Shear reinforcement must be present in the web of the beams in order to resist the upthrust force exerted on the shoulder of the top flange of the precast beam by the insitu concrete infill and prevent end splitting.

In the case of web and interface connection provided with sufficient shear reinforcement, the cracks develop more gradually in the precast units away from the connection and near the ends of the solid infill diaphragm. Due to the pesence of shear reinforcement, applied moment was transformed into couples in the connection, i.e. tensile forces in the links balanced by compressive force exerted on the flange of the beam.

The reinforced connection using the infill diaphragm is seen to be effective in transferring the sagging moment from one span to another. An extensive series of tests is now under way to investigate the effect of the joint parameter, and to improve understanding of the moment transfer mechanism.

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EVALUATION OF HONEYCOMB CONCRETE

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ABSTRACT. This paper presents two cases where concrete mixtures placed through congested reinforcing steel resulted in segregation of coarse aggregate and honeycombing. The first case history involves construction of heavily reinforced girders. The mix design used contained 1 in. (25 mm) maximum size aggregate and a net spacing between No. 11 reinforcing bars which was less than the minimum allowed by ACI. Upon form removal severe honeycombing was revealed. The paper discusses the evaluation methods. The second case discusses the causes of honeycombing and rock pockets which occurred in structural elements for a large power plant. The cause of the rock pockets was severely congested reinforcing steel and an inappropriate concrete mix design with respect to coarse aggregate size.

Keywords: Consolidation, Honeycomb, Rock Pockets, Maximum Nominal Size Aggregate, Congestion, Slump, Flowing Concrete, Vibration.

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preparation of repair documents. Mr. Stejskal's expertise includes use of non-destructive testing such as ground motion (seismic) detectors, radar and ultrasonic test equipment.

INTRODUCTION

Honeycombing is the result of inadequate consolidation, mix segregation and/or loss of paste or mortar between forms. Improper consolidation of concrete can severely compromise the durability and integrity of concrete members. Honeycomb concrete is usually characterized by the absence of cement paste or mortar leaving loosely bonded coarse aggregate particles with a "honeycomb" appearance.

Another term used in describing underconsolidated concrete is "rock pocket." Clusters of coarse aggregate with little if any paste or mortar result when concrete placement is attempted in areas of congested reinforcing steel. In fact, any situation where concrete flowability or consolidation is impeded can compromise the achievement of potential concrete strength and durability [1].

Experience has shown that for congested structural elements, honeycomb concrete can occur when concrete mixtures are used which contain aggregate sizes equal to or greater than the actual steel spacing. Additional contributing causes are poor member design with respect to constructability, poor placement and consolidation techniques, form leakage and others.

This paper presents two cases where concrete mixtures placed through highly congested reinforcing steel resulted in segregation of coarse aggregate causing formation of rock pockets or honeycombing.

CASE HISTORY NO. 1

Office/Warehouse Building, Honeycombing of Beams and Girders

The construction of a new office/warehouse complex in the United States included heavily reinforced concrete. The floor slab of the building was typically supported by an integrally cast beam girder system. The girders contained up to two rows of twelve No. 11 bars both top and bottom and No. 4 stirrups which were spaced as close as 3 in. (75 mm). The girder cross section was reduced from 42 in. (1.05 m) wide down to 36 in. (0.9 m) at the bottom. The tightest bar spacing at the bottom of the girders was 1.28 in. (32 mm) between bars with 1–1/2 in. (37.5 mm) cover. This bar spacing is a "best case" scenario in that this does not consider splices, embedments or normally acceptable construction tolerances. This bar spacing was nearly impossible to provide in the field as shown in Figure 1 which shows No. 11 bars routinely in contact. The 4,000 psi (27.5 MPa) concrete mix design contained a 1 in. (25 mm) maximum aggregate size and a slump limit of 4 in. (100 mm).

The concrete floor and support system was placed monolithically using a metal pan forming system. When the metal forms were removed extensive honeycombing was found.

Presented below is the evaluation approach and summary of findings.

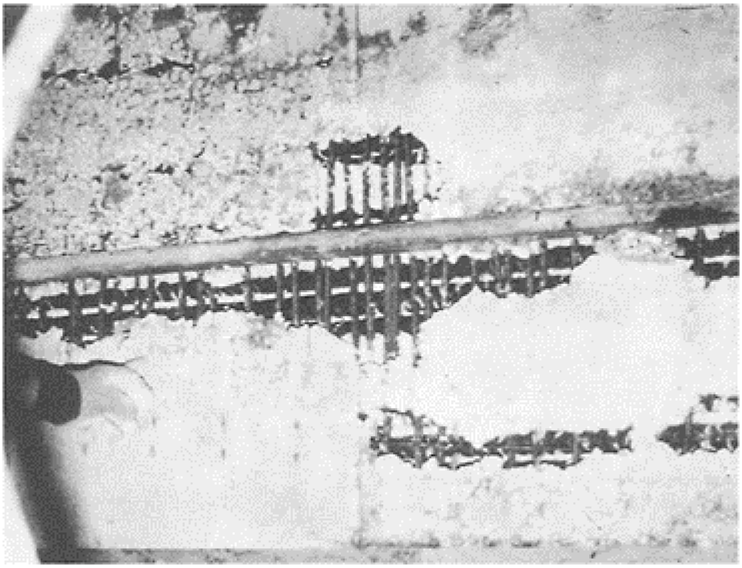

Figure No. 1—Typical Girder Honeycomb

The Evaluation Approach

An evaluation of 30 honeycombed concrete girders included the following steps:

1. Visual inspection of all concrete from the slab underside.
2. Hammer sounding of all concrete and recording distress observed such a honeycombing, cracking, delamination or obstructions which prevented surface inspection.
3. Coring of selected beams as well as small diameter probe holes to determine the extent of honeycombing within the girders.
4. Pulse-velocity measurements of cores, test cylinders and in-place concrete.
5. Laboratory testing of concrete cores to determine compressive strength.

Findings of the Evaluation

Findings of the visual inspection and hammer sounding were recorded to estimate repair quantities.

Pulse velocity measurements were taken perpendicular to the member length along the girder edges on a 1 ft (0.3 m) grid above the bottom reinforcing steel. Velocities ranged from 13,900 fps (4,237 m/s) to 14,200 fps (4,328 m/s). These readings compared to compressive strength/pulse velocity data from concrete cores and cylinders appeared to indicate that the concrete would exceed the specified compressive strength.

All concrete test cylinders exceeded the specified strength. Pulse-velocity measurements taken on project cylinders averaged 15,220 fps (4,633 m/s) which was slightly higher than cores. A visual examination of the cores drilled through mid-depth of six selected girders found the concrete to be well consolidated and exhibiting uniform aggregate distribution. This finding indicated that mix segregation occurred toward the bottom of the beam where it narrows from 42 in. (1.05 m) to 36 in. (0.9 m). Compressive strength of the cores were found to comply with project specification requirements. The pulse velocity measurement taken of the cores and girders indicated a standard deviation of 273 fps (83.2 m/s) which was judged to represent uniformly consolidated concrete. The overall close grouping of compressive strengths and pulse velocity measurements indicated relatively uniform concrete quality in the areas tested.

CASE HISTORY NO. 2

Honeycombed concrete was also a problem in structural members of a powerhouse under construction in the U.S. The honeycombing was a direct result of highly congested reinforcing steel in the walls and girders, especially in the vicinity of wall penetrations. Figure 2 shows congested steel at the wall penetration. The high concentration of reinforcing steel resulted in a less than adequate steel spacing to allow flow of the concrete. The nominal maximum aggregate size exceeded the reinforcing bar spacing resulting in honeycomb of the mix. Stiff or low slump concrete required by project specifications also made concrete placement difficult resulting in honeycombing or rock pockets as shown in Figure 3. Even the vibration of a flowing consistency concrete cannot remedy this situation because the coarsest aggregate cannot pass through the reinforcing steel. Additional factors that caused rock pockets were:

- The clear cover was insufficient for the aggregate size violating ACI 318 provisions (aggregate bridging preventing flow of concrete between steel reinforcement and form).
- Reinforcing steel spacing was less than the minimum one bar diameter as required in ACI 318.
- Inadequate spacing between parallel layers of reinforcing to accommodate aggregate size.
- Class C splices were required (for No. 11 bar this would require 4 ft-6 in. (1.4 m) of lapping).

Factors Contributing to Honeycombing (Rock Pockets)

The main cause of honeycombing was that the required maximum size of coarse aggregate was 1 in. (25 mm); thus such aggregate would not pass through spacing between steel bars and bars and formwork which was sometimes are small as 3/4 in. (19 mm). In addition, low slump concrete was required. The average slump specified had to be below the working slump limit of 3 in. (75 mm). Thus, some slumps were as low as 2 in. (50 mm) which while meeting specifications, would be almost impossible to place through such congested reinforcement.

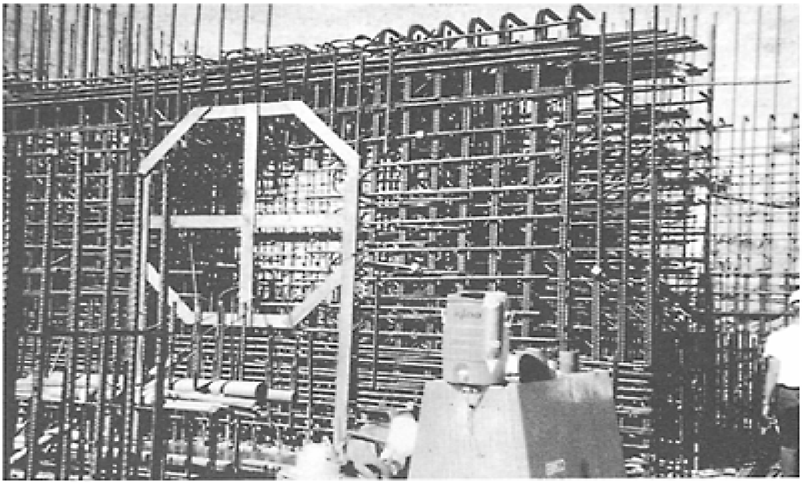

Figure 2—Wall penetration; note reinforcing steel conjestion near penetration.

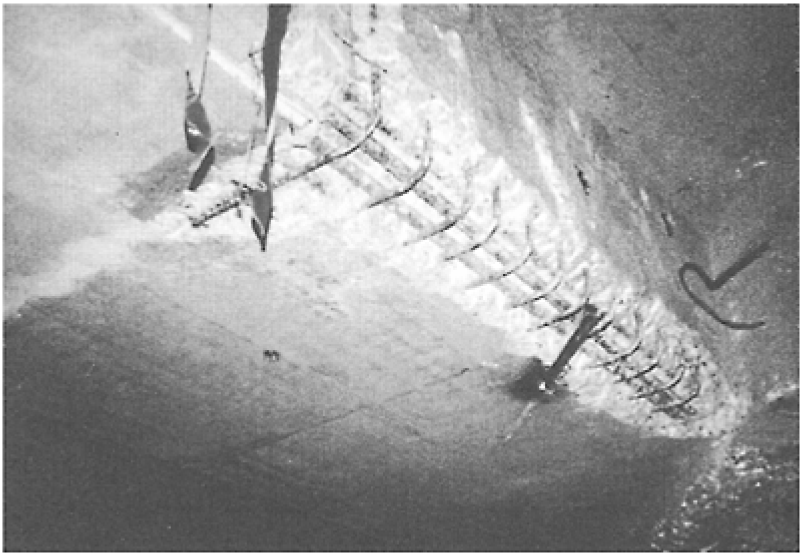

Figure 3—Congestion due to inadequate bar spacing resulting in rock pocket.

Girder A

Girder A was designed with the following parameters:

Width—66 in. (1.65 m).
Cover—1–1/2 in. (38 mm).
Main Reinforcement—3 layers of 16-No. 11 bars.
Stirrups—6 No. 6.
Spacing between layers of reinforcing bars: 1–1/4 in. (31 mm) (with a 1/4 in. [6 mm] tolerance the minimum clear spacing is 1 in. [25 mm]).
Nominal Maximum Size Aggregate (NMSA)—1 in. (25 mm).

Besides the problem of the aggregate size being too large for the parallel reinforcement, the average clear spacing between bars was 2.4 in. (61 mm). Taking into consideration that laps are required in this area and the lengths of these laps are approximately 4 ft-6 in. (1.4 m), this results in a clear spacing between bars of 1 in. (25 mm). ACI 318 Code requires that the clear spacing between bars be greater than or equal to d_b, the bar diameter.

Girder B

Girder B was designed to contain the following elements:

Width—48 in. (1.2 m).
Cover—1–1/2 in. (38 mm).
Main reinforcement 12—No. 11 bars.
Stirrups—4 No. 5.

The resulting average clear spacing between steel reinforcement was 2.32 in. (59 mm). This steel arrangement met the Code. However, intersecting with this member is a column that contained a curtain of No. 4 ties and No. 10 vertical bars. The net result is that clear spacing between bars was reduced to 3/4 in. (19 mm). Besides failing to meet code requirements for db, the aggregate size becomes a more significant overriding factor in the formation of rock pockets. By Code the NMSA should not have been greater than 9/16 in. (13 mm).

RECOMMENDATIONS

ACI Committee 309, Consolidation, has published several documents concerning how to construct honeycomb free concrete. ACI 309.3R [2] Guide to Consolidation of Concrete in Congested Areas, presents examples of what is considered to be congested reinforcement. The Guide simply states:

> "Congested areas are those in which the reinforcing steel, embedments, boxouts, prestress ducts and anchorages, or configurations and form shape make concrete placement and consolidation difficult to achieve."

Where requirements dictate that highly congested sections are needed, the designer and contractor should meet to resolve constructability issues.

We have shown through these two case histories that honeycombs or rock pockets could have been avoided using some simple rules and common sense. The following list of recommendations is presented to reduce or eliminate the occurrence of honeycombing.

- Re-evaluate the member design, especially overall dimension and connection to other members.
- Consider tolerances for cover and bar spacing in the design stage.
- Use alternate splicing methods—mechanical vs. laps.
- Consider appropriate mix design especially aggregate size, admixtures, appropriate workability and thorough consolidation.
- Maintain form tightness to prevent paste or mortar leakage.
- Ensure that consolidation methods can be effectively accomplished.

The recommendations provided in this paper and those detailed in ACI 309.3R should be followed to reduce the risk of costly repairs associated with these defects. The associated costs for using modified mixtures, smaller coarse aggregate and/or redesigning the member will more then offset the cost of remediation.

Using a concrete mix containing too large a coarse aggregate for bar spacing, cover, and steel density contributes to bridging of aggregate particles and segregation. Stiff harsh mixes require more compactive effort for proper consolidation and are difficult to pump.

Another consideration to offset the effects of densely placed reinforcement is to attain access for vibrators to achieve proper consolidation. If restrictions on the spacing and density of the steel are so difficult it may also be necessary to employ smaller diameter vibrators in conjunction with the normal size vibrators to consolidate the concrete. In extreme cases, form vibrators may be helpful in conjunction with internal vibrators.

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FLEXIBLE REINFORCED CONCRETE BEAMS ON ELASTIC RUBBER FOUNDATION MODEL

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ABSTRACT. Concrete slabs required for roads, runways and heavy industrial pavement are thick and consequently stiff, and crack when deflected by ground subsidence, internal shrinkage or temperature gradient in combination with traffic loading. By dividing thick concrete into a number of thin independent layers a laminated slab can be constructed with significantly increased flexibility. This type of slab would settle under self weight and retain better contact with the sub-grade than the equivalent solid section. It would however, remain flexible when loaded by traffic and thus, place higher stresses on the soil below. However, in the proposed model, to retain a proportion of the stiffness of the full depth of concrete, the lamination are bonded together with bitumen. The bitumen would creep during a period of prolonged subsidence allowing the slab to deform with the sub-grade, but would be stiff when rapidly loaded by traffic. This paper contains details of a laboratory investigation of the behaviour of 6m long, 200mm deep and 150mm wide reinforced concrete laminated beams on three different support stiffnesses (1, 3 and 9 thickness of rubber pads) under static and rapid loading. The behaviour of laminated beams are also investigated with an artificial settlement profile under the simulated rubber pad supports. In this paper the effect of lamination is shown by comparing the deflection of laminated beams with that of equivalent solid beams under the same test configurations.

Keywords: Rubber pad, Laminated, Solid, Rapid-hardening Portland cement.

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INTRODUCTION

It is known that a beam composed of laminations placed freely upon one another deflects more under a given load than does a solid beam of the same cross section under the same loading conditions. The difference in deflection is caused by the fact that in the solid beam all longitudinal planes can support shear, whereas a freely laminated beam can not resist shearing load on the planes that separate the individual laminations. This paper discussed a laboratory investigation which was performed to determine the response of a concrete laminated beams bonded together with a means of a thin layer of bitumen. In order to reduce the internal stresses while maintaining the benefits of a thicker concrete pavements.

This inability to resist horizontal shear has led the use of various devices such as riveting, welding, or bolting, to connect the laminations so that the laminated beam can be made to act similarly to a solid beam.

Due to the viscosity of the bitumen layer the resulting laminate acts as a whole when suddenly loaded by the traffic, and as individual thin flexible laminate when slowly internally stressed by thermally induced volume changes in the concrete, or loss of support from the soil below.

SUB-GRADE MODEL

The model sub-grade material was required to simulate two types of behaviour.

a) a relatively high recoverable deflection under load, due to weak sub-grade material.

b) a deflection due to long term irrecoverable subsidence of the sub-grade independent of the applied load.

Previous investigations into the behaviour of layered systems have widely adopted support conditions using soil [1,2]. It has been generally found however, that a uniform condition is difficult to achieve and in any case ensuing moisture changes lead to variation in support. Other investigations used a Winkler foundation, consisting of a set of independent springs arranged in a circular pattern as the support for a series of tests on bituminous beams and slabs. Other investigators adopted support conditions using closed-cell expanded natural and solid rubbers used for support layers with cemented and bituminous materials [3,4,5 and 6].

This investigation is concerned with the study of the behaviour of laminated beams on an elastic rubber foundation model under static and rapid (traffic) loading.

BITUMEN AS BONDING LAYER

In the proposed concrete laminated model the bitumen is required to have;

a) maximum stiffness under rapid loading (traffic),
b) minimum stiffness (maximum flexibility) under longer time of loading,

i) daily thermal changes,
ii) seasonal thermal changes,
iii) sub-grade subsidence.

The thermal stresses can be as great as those due to traffic.

Due to criteria in (a) and (b) two different type of bitumen have been investigated;

1) pitch type
2) penetration type

Initially pitch type bitumen with penetration grade of 1 at 25 degree centigrade, softening point of 85 degree centigrade and PI value of −1.6, was investigated. The stiffness, calculated from Van der Poal Nomograph, showed that this type of bitumen gave maximum stiffness under very short loading time and minimum stiffness at longer loading time when compared with other bitumens.

However, there are problems with pitch type bitumen of adhesion to other materials, health and safety handling problems and great susceptibility to temperature changes.

From the above investigations and literature studies it was decided a penetration grade bitumen with 15 pen and softening point of 70 degree C, would be preferable for the proposed model and consequently it was decided to investigate the properties of that particular bitumen, for more details see [7].

To illustrate different loading time, typical rates of loading have been taken. Assuming a vehicle passing over the slab at 60 Km/h would take 0.2 of a second to load and unload.

PROCEDURE

Beams preparation

Initial preparation started with laying out the rubber pads to the required dimensions and pattern. The rubber foundation for the beams considered of a row of 21 positions with either one 6mm, or 3×6=18 mm, or 9×6=54 mm rubber pads each 150×80mm in plan at pre-determined intervals. Steel shuttering was used for the sides, which was bolted to supporting frames using an angle section. When the mould had been erected and positioned relative to the reaction frame, 2 number 8mm diameter high tensile reinforcement bars were laid in position, (at centre). The load footprint used throughout this investigation was a rectangular 150×100 mm steel plate 30 mm thick which was bedded on a layer of plaster to give an even load distribution. Load was transmitted via a calibrated proving ring with 50 kN capacity from a manually operated screw jack with 50 kN capacity, for application of a constant rate of deformation.

The laminated beams were also subjected to different constant load applications for creep measurement by means of a lever arm method. Data were recorded by observation of dial gauges over the length of the beams at different time periods up to 24 hours. In each case on un-loading the procedure was the same sequence as for loading.

Concrete mix details

Table 1 gives details of concrete mix for beams studies, as proportioned by weight. The mixes had a nominal aggregate/cement ratio of 3.75. Rapid hardening Portland cement (RHPC) was used throughout. Aggregate were Thames Valley irregular gravels and sand, and these materials were oven dried prior to use.

Table 1 Concrete mix details

Proportion by unit weight (kg/m^3)*	**Nominal aggregate/cement ratio (3.5:1)**
Cement (OPC)	500.00
Coarse aggregate (SSD) (maximum size 20 mm graded) (greenwich, Marine)	1330.00
Sand (zone M), (SSD) (Greenwich, Marine)	430.00
Water	164.00
Conplast 509 (l/m^3)	1.0

* Weight required to produced 1 m^3 of concrete

In laminated beams, after casting, the beam was left to cure for 3 days in a wet condition and the bitumen layer with 1.5mm thickness (as discussed in more details in Ref 7), was commenced on the fourth day. At this stage the side moulds were extended up for the top concrete layer, as for the bottom layer. Twenty four hours after casting the top layer the mould was stripped down and the concrete cured as for the bottom layer for three days and the final preparation for testing commenced on day four. This include the positioning and fixing of transducers and dial gauges on the beam side near the lamination level and on the top surface, see Figure 1 for general view.

For Solid beam the reinforcement bars were laid in position at 50 mm and 150 mm depth to comply with configuration of laminated beams. After casting the same procedure was followed as for laminated beam.

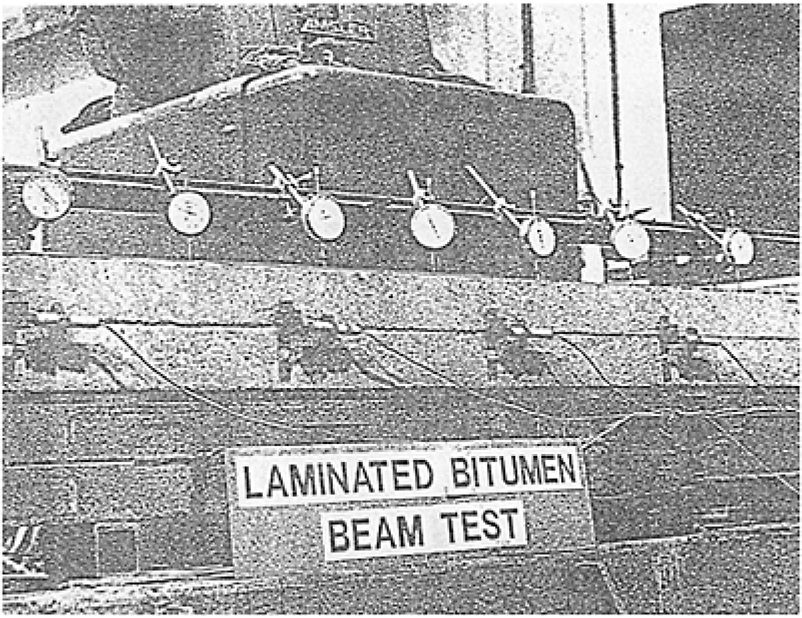


Figure 1 General view of two layer laminated beam

TEST METHODS

The function of a reinforced concrete laminated pavement is to satisfactorily carry a given traffic load for a specified short period of time and to deform with ground subsidence over a much longer period. To design a pavement which will properly perform these functions a knowledge of the stresses and deflections in the pavement system is necessary.

To investigate the basic problem and gain better understanding of the response, the system was represented by laminated reinforced concrete beams on an elastic sub-grade.

The behaviour of laminated reinforced concrete beams on three different support stiffnesses (1,3 and 9 pad thicknesses) under various loading was investigated.

In rapid loading tests the beams were tested in about 0.2 second. To elaborate on this further, 0.2 second is the time taken to apply the full load on the beams. This also represents a vehicle travelling at a speed of about 60km/h. The increased deflections after this time (0.2 sec) are regarded as delayed shear in the bitumen and as creep which is due to the rubber support and possibly the concrete itself. For constant static loading tests the load was lowered slowly on the beam and left for 24 hours. Hence, the deflection profiles were recorded during this period at 1, 10 and 1440 minutes (24 hours).

RESULTS AND DISCUSSION

Beams were tested using various constant rates of central deflection (recording the load necessary to maintain the constant rate) and also by applying various constant loads for chosen durations (recording the central deflection, and deflection profiles along the beam). Constant load tests were repeated on a solid beam with no lamination. The main variable in all cases were central deflection (and deflection profile), load and time, (either as duration of loading or rate of deflection).

All beam tests were carried out on three different model sub-grade stiffnesses (soft, medium and stiff consisting of 9, 3 and 1 thicknesses of rubber pad).

Behaviour of the solid and laminated beams

Table 2 gives the deflections observed at four different loading times with three different sub-grade stiffnesses.

Upon comparing the results for the solid and laminated beams, it can be seen that the loading time effect on the solid beam stiffness is small and is due to properties of the rubber pads and possibly the concrete itself. The results for the laminated beams conclude that the shear stiffness of the beams during traffic loading is not seriously affected by lamination with bitumen, see Figure 2. The bitumen lamination is stiff during rapid loading (traffic loading) and there is little difference in response between the solid and laminated beams. It is also shown that with time the central deflection increases but the width of the deflection bowl decreases. Comparing the hard and soft sub-grades, on the soft sub-grade the initial deflection is greater and hence the laminated beam is initially stiffer.

Behaviour of laminated beam with partial settlement

The principal aim of this experimental investigation was to determine the performance of laminated reinforced concrete beams, when supported on rubber pads simulating a sub-grade with an artificial settlement profile of 0.35mm at centre or at the ends of the laminated beam. The settlement profile which was adopted under the beam was taken from the deflection profile of the beam with normal contact (with all the pads taking approximately the same proportion of beam's

Table 2 Central deflections and test configurations

			Loading	Time		
Beam Type	**No of Pads**	**Load(KN)**	**0.2 sec**	**1 min**	**10 min**	**24 hours**
	1	**2.20**	**0.040**	**0.050**	**0.060**	**0.080**
Laminated	**1**	**5.45**	**0.105**	**0.110**	**0.120**	**0.130**
	1	**7.55**	**–**	**0.154**	**0.164**	**0.174**
	1	**2.20**	**0.040**	**0.043**	**0.51**	**0.058**

Solid	**1**	**3.65**	**0.070**	**0.080**	**0.085**	**0.093**
	1	**5.45**	**0.010***	**0.106**	**0.110**	**0.116**
	3	**2.20**	**0.069**	**0.086**	**0.092**	**0.110**
Laminated	**3**	**3.65**	**0.113**	**0.130**	**0.140**	**0.166**
	3	**5.45**	**0.150**	**0.180**	**0.200**	**0.220**
	3	**2.20**	**0.054**	**0.067**	**0.070**	**0.094**
Solid	**3**	**5.45**	**0.120**	**0.147**	**0.153**	**0.167**
	3	**7.55**	**0.108***	**0.163**	**0.171**	**0.184**
	9	**1.10**	**0.100**	**0.125**	**0.143**	**0.165**
Laminated	**9**	**2.20**	**0.178**	**0.235**	**0.250**	**0.335**
	9	**3.65**	**0.260**	**0.400**	**0.430**	**0.490**
	9	**1.10**	**0.094**	**0.105**	**0.110**	**0.137**
Solid	**9**	**2.20**	**0.143**	**0.150**	**0.160**	**0.180**
	9	**3.65**	**0.200**	**0.211**	**0.230**	**0.250**

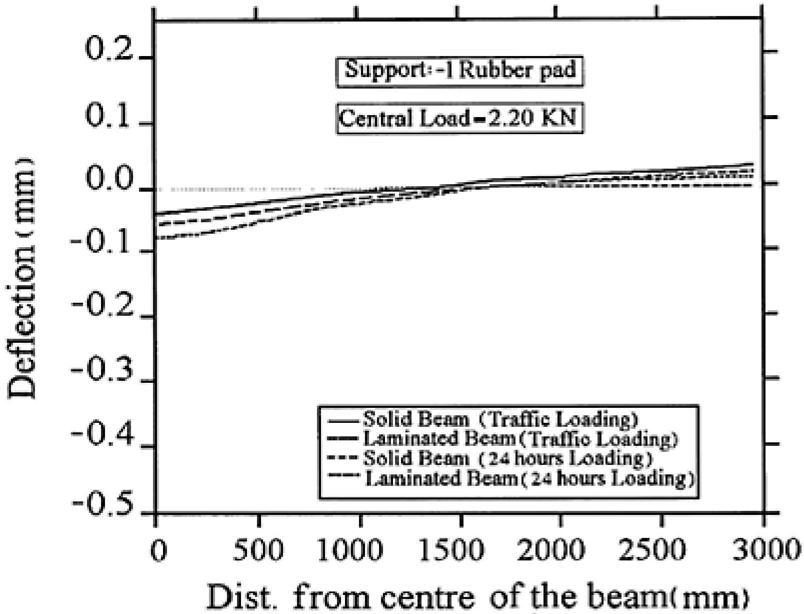
* Estimated

weight), for method of creating the settlement profile see [7]. From Figure 3, 4 it can be seen that the surface deflections are more or less the same under traffic loading. This shows no or very little loss in stiffness of the laminated beams. In the slow loading the surface deflections were greater (10%) over 24 hours when artificial settlement was introduced under three rubber pads and 14% greater when artificial settlement was introduced under 9 rubber pads. The foregoing discussion has presented an evaluation of laminated beam response under a known settlement profile. In particular it has been shown that the behaviour of the laminated beam supported over a subsidence sub-grade model is in agreement with the predicted behaviour, that the beam will tend to deform with the subsidence, so that deflection under load is relatively unaffected by the ground movement.

It is also shown that, with longer loading time the deflections along the laminated beams increased significantly especially on softer sub-grade.

CONCLUSIONS

When designing pavement it is assumed that the slab is in contact with the soil and the slab distributes the load directly to the sub-grade. If the soil subsequently deforms, as it often does, this leaves the slab acting as a bridge between high spots. The thicker the slab the more rigid it is and hence the greater the span between the



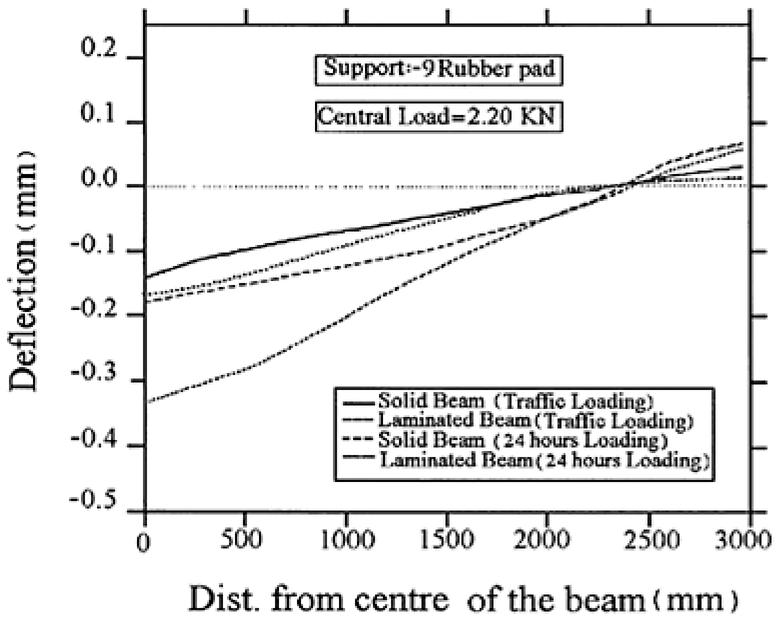


Figure 2 Comparative deflection profiles of solid and laminated model beams with varying load duration and sub-grade stiffnesses.

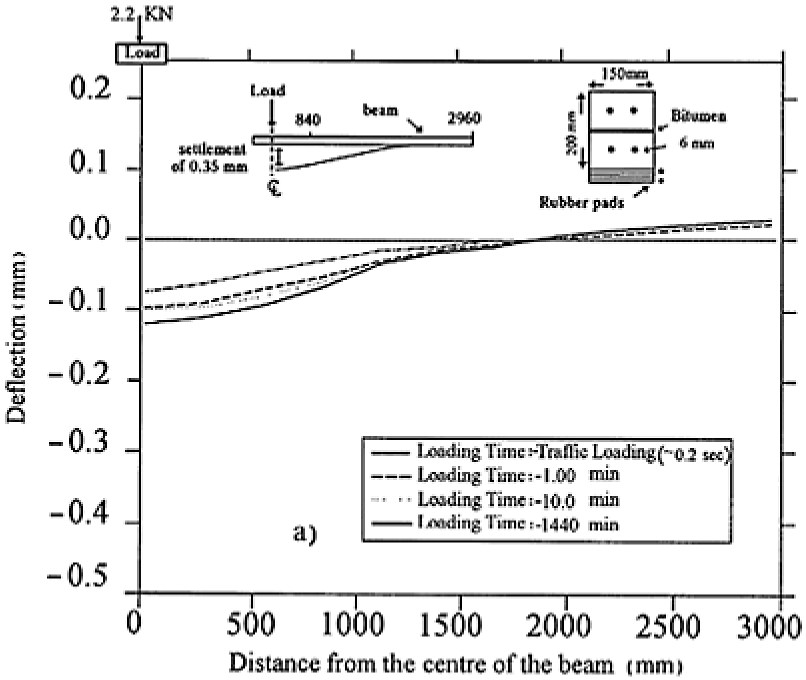


Figure 3 Laminated beam support on 3 rubber pads sub-grade model with settlement profile gradually built over its length (total=0.35 mm) at centre.

high spots. The results showed that this was not the case with laminated model beams, which retained its flexibility due to the laminations and maintained contact with ground between high spots. This enabled the traffic load to be transmitted directly to the sub-grade without over stressing the pavement, while maintaining the benefit of the thicker concrete pavement. The other principal findings arising from this work are as follows;

1. The laminated beam tests showed that the beams retained 75% of the stiffness of a solid beam of the same cross section when loaded rapidly. In long term loading the laminated beam stiffness was reduced by 30%. The experiments also showed that, the stiffness of laminated beams under the same loading system increased with reduced support or sub-grade stiffness (thickness), whereas with the solid beam this effect was insignificant.

2. The experiments showed that in the laminated model, the internal stresses are reduced as a result of laminating the thicker concrete pavement with bitumen which allows each lamination to slowly change length with temperature, reducing ground friction.

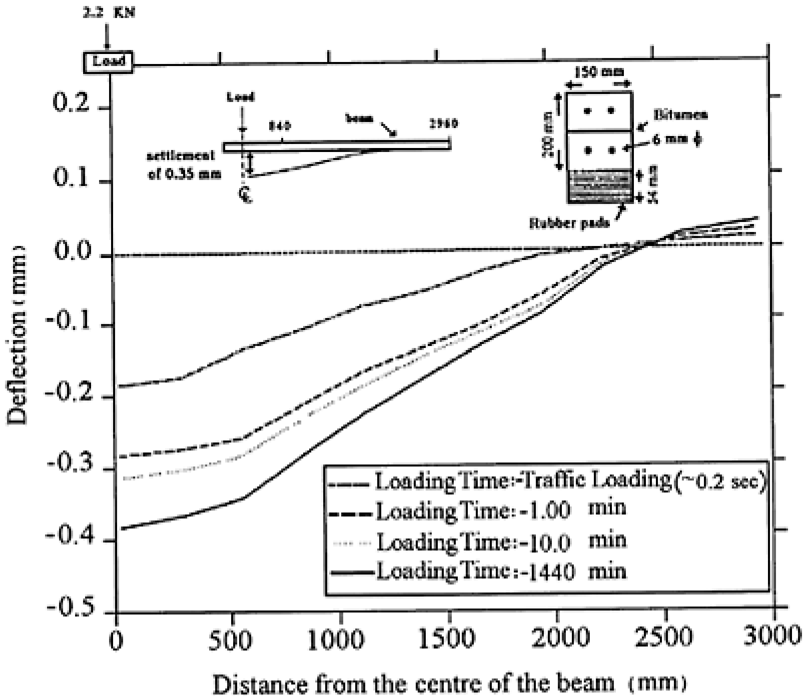


Figure 4 Laminated beam support on 9 rubber pads sub-grade model with settlement profile gradually built over its length (total=0.35 mm) at centre.

3. A simulated elastic sub-grade with adjustable properties was established.

4. Experiments with an artificial settlement profile created under the rubber pad supports showed that, the laminated beams under rapid loading would retain almost the same stiffness as before subsidence. With longer loading time the deflections along the laminated beams increased significantly especially on softer sub-grade.

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PORTAL A-FRAME VS PORTAL H-FRAME STRUCTURES A CHOICE BASED ON TOTAL COSTS

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ABSTRACT. In a substation, the conductors of a three-phase transmission line are attached to an anchor structure. That can be either an A-frame or an H-frame portal structure. The conductors are supported at the frame ends and at the center of the rigidly connected transverse beam at a height of 15 meters for a 120 kV line. In addition to the conductors, a ground wire is supported at one end of the structure, at a height of 18 meters. Using an H-frame portal structure, each column must be anchored to a caisson foundation because of the large base bending moments, whereas for an A-frame portal structure with legs six meters apart, isolated superficial foundations would be suitable. This paper compares the costs of the two solutions including all the material and construction costs. It is shown that the type of foundation needed has a major impact on the total costs. The analysis is done for a barely over-consolidated clay which allows a net bearing pressure of only 15 kPa. This case was encountered at the Groulx substation where this study supported the choice of an A-frame portal structure.

Keywords: Substation, Transmission lines, Sags, Tensions, A-frame portal structure, H-frame portal structure, Caisson foundation, Superficial foundation, Soil pressures, Bearing capacity, Soil reaction modulus, Costs.

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INTRODUCTION

Different types of structures or towers can be used to anchor the 3-phase transmission lines at the entrance of a substation. For esthetical reasons, tubular section structures are usually preferred over lattice-type structures. The three types of structures most commonly used at Hydro-Quebec and elsewhere include a set of three isolated poles, an A-frame portal structure or an H-frame portal structure. The choice of one type over the other depends on several factors including the horizontal angle of arrival of the lines, the layout of the electrical equipment around the structure, space limitations and costs. When the oncoming angle of transmission lines is less than about 15°, portal type structures can be used. If so, the choice between an A-frame and an H-frame structure is mainly dependant on costs.

The first part of this paper gives the loading criteria for the 120 kV transmission lines arriving at the Groulx substation. The second part presents the designs of an A-frame structure with its superficial foundations and an H-frame structure with its caisson foundations for a soft clay soil. Total costs are detailed and, finally, a comparison of the two solutions is provided.

LOADING CRITERIA

The loading conditions on the structures are governed by the sag and tensions of the transmission lines which are dependant on the climatic conditions prevaling at the site. Wind and ice loads were obtained from Hydro-Quebec technical specification SN-40.1 [1]. They are limit loads corresponding to an acceptable risk of being exceeded in the useful lifetime of the lines. The 120 kV conductors have a diameter of 27.8 mm and a unit weight of 14.9 N/m. The ground wire has a diameter of 11.0 mm and a unit weight of 5.7 N/m. The span of the lines is 205.5 m. The sags and tensions for the different loading conditions are given in Table 1. The angle of the oncoming lines is 10°. Another loading condition to consider includes earthquake forces, which in the present case do not govern.

Table 1—Sags and tensions

	Loading conditions							
	45 mm ice, 0°C 0 kpa wind		**20 mm ice, 0°C 0.65 kpa wind**		**0 mm ice, −10°C 1.2 kpa wind**		**0 mm ice, −30°C 0 kpa wind**	
Cable	Horizontal tension (kN)	Sag (m)	Horizontal tension (kN)	Sag (m)	Horizontal tension (kN)	Sag (m)	Horizontal tension (kN)	Sag (m)
Conductor	72.2	7.7	51.1	6.2	38.0	5.1	24.4	3.2
Ground wire	48.0	8.3	31.2	6.8	15.6	4.9	9.0	3.3

DESIGN OF PORTAL A-FRAME STRUCTURE

Steel structure and superficial foundations

The A-frame portal structure shown in Figure 1a is composed of hollow square tubular sections taken from the Canadian Handbook of Steel Construction [2].

The net soil bearing capacity is 15 kPa for permanent loads with a safety factor (S.F.) of 3 for a maximum settlement of 25 mm or 22.5 kPa for ultimate loads with S.F.=2. The soil density (γ) is 17.5 kN/m^3. Because of the very weak overconsolidation of the clay, the footings of the two columns in compression would have to be either on piles or almost fully compensated. The latter solution was retained and the granular backfill was replaced with styrofoam. The details of the compression and uplift foundations are shown in Figure 1b along with the permanent and ultimate loads. The ultimate loads for the compression foundations are governed by the 45mm ice condition. For the uplift foundations, the 0.65 kpa wind-20 mm ice condition is critical. The pressure distribution under the footings is given by the following general expression,

$$q=P/A\pm M_x/S_x\pm M_z/S_z$$

where q is the soil pressure, P is the vertical load, A is the effective footing area, M_i and S_i are the bending moment and the footing effective section modulus about i respectively. For the compression foundations, the permanent loads give q_{max}= 30.5 kpa vs 46.5 kpa allowable and q_{min}=19.5 kpa>0 (no overturning). The ultimate loads give q_{max}=54 kpa which is the allowable pressure (22.5+17.5×1.8) and q_{min}=21 kpa>0 (no overturning). Sliding and torsion are resisted by the Rankine passive pressure at the level of the footing using a coefficient, Kp, of 3. The maximum pressure applied is 48 kpa resulting in a S.F. of 2 not considering the base friction. The uplift foundations are treated in the same manner and will not be discussed here. The maximum bending moment and shear force in the footing are respectively 88 kN.m/m and 150 kN/m. The maximum bending moment and shear force in the beams are respectively 553 kN.m and 353 kN. The concrete design is according to CSA standard CAN3-A23.3 included in the PCA Concrete Design Handbook [3].

Total costs

The mass of the steel structure is approximately 11 500 kg including anchor bolts. The unit cost of fabrication is $3.00/kg, resulting in a structure cost of $34 500. The volume of concrete for each footing is 12.7 m^3. The unit cost of construction is $700/m^3 which includes the concrete, the forms, the reinforcing bars and the styrofoam filling (or granular backfill). The cost of the four superficial foundations is then $35 560 and the total cost amounts to $70 060.

DESIGN OF PORTAL H-FRAME STRUCTURE

Steel structure and caisson foundations

The portal H-frame structure illustrated in Figure 2a is fabricated with twelve-sided folded plates welded longitudinally and transversally. All sections are 8 mm thick. The design of the members follows the specifications of the ASCE report no. 72 [4].

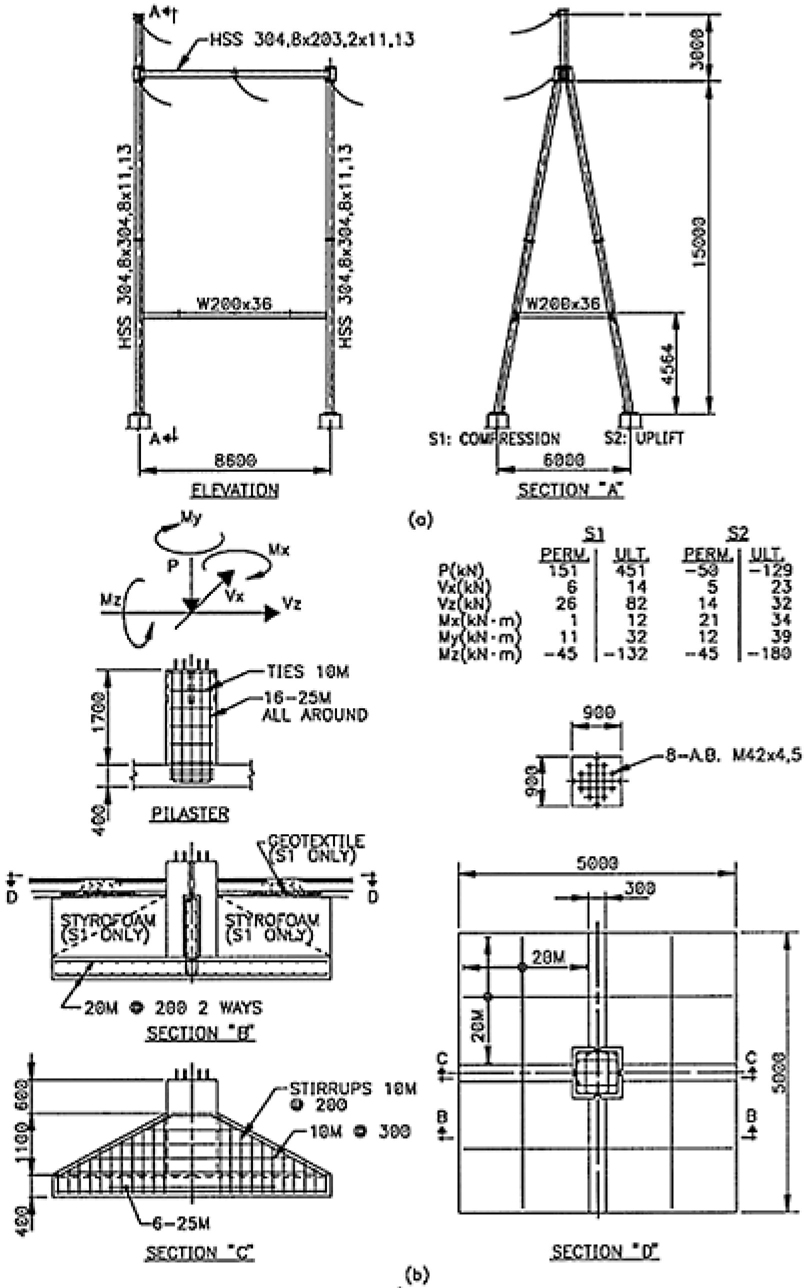
The design of the caissons illustrated in Figure 2b is based on the soil horizontal reaction modulus (Ks) to lateral pressures, which in turn is dependant on the undrained shear strength of the clay (Cu), which is 30 kpa at the critical depth (z_c= 3D) of 5.5 m. The reaction modulus for a 0.3 m wide caisson (Ks_1) as a function of Cu is taken from Marche [5]. The variation of Ks with depth (z) is expressed by the exponential form given below and takes into account the diameter of the caisson (D) and a surface correction,

$Ks(z)=Ks_c\ (z/z_c)^{0.3}$ and $Ks_c=Ks_1\times0.3/D$ where Ks_c is the reaction modulus at z_c

One way to analyse a caisson is by replacing the soil with springs having stiffnesses equivalent to the Ks values. The numerical model is shown in Figure 2c, along with the loads obtained with the 45 mm ice critical condition. The required length of the caisson is determined so that its maximum rotation is limited to 0.2° and the allowable pressure of the soil is not exceeded. The allowable pressure (Pa), is fonction of the creep pressure (Pf) and the limit pressure (PI) taken from Marche [5] with appropriate safety factors. Hence, at z_c, $Pl_c=9\times Cu$, $Pf_c=Pl_c/1.5$ and $Pa_c=Pf_c/2=90$ kpa. The variation of Pa with depth is given by,

$Pa(z)=Pa_c(z/z_c)^{0.3}$ for $z\geq z_c$ and $Pa(z)=Pa_c(2+7\ z/z_c)/9$ for $0<z<z_c$

The design of the caissons is according to the Handbook of Steel Construction [2] and the PCA Concrete Design Handbook [3].



	S1 PERM.	S1 ULT.	S2 PERM.	S2 ULT.
P(kN)	151	451	−50	−129
Vx(kN)	6	14	5	23
Vz(kN)	26	82	14	32
Mx(kN·m)	1	12	21	34
My(kN·m)	11	32	12	39
Mz(kN·m)	−45	−132	−45	−180

FIGURE 1 A-FRAME PORTAL STRUCTURE WITH SUPERFICIAL FOUNDATIONS

Total costs

The mass of the steel structure is approximately 9 200 kg including the anchor bolts. The unit cost of fabrication is $3.70/kg resulting in a cost of $34 040 for the steel structure. The two steel caissons are 13 mm thick and have a total mass of 24 800 kg and their cost amounts to $91 760. The unit cost of construction is set at $1 200/m which includes the mobilisation of the crew and the equipment, hammering the caissons, cleaning the inside, field welding, the concrete and the reinforcing bars. For a total caisson length of 42.6 m, the cost of construction is $51 120. The cost of the caisson foundations amounts to $142 880 producing a total cost of $176 920 for the structure and the foundations.

CONCLUSIONS

For practical purposes, it can be assumed that the total cost of steel is the same for an H-frame or an A-frame portal structure, considering that the latter is only 1.4% more expensive. However, for the weak soil considered here, the superficial foundations are four times less expensive than the caisson foundations. The total cost of an H-frame structure with caissons is then twice and a half more expensive than an A-frame structure with spread footings. Consequently, the latter becomes clearly the best choice. In addition, some electrical equipment can be mounted on the A-frame structure, therefore saving ground space and equipment foundations. As the soil quality increases, it can be expected that the cost margin between the two types of foundations will narrow. The rate of decrease would need to be looked into further.

ACKNOWLEDGEMENTS

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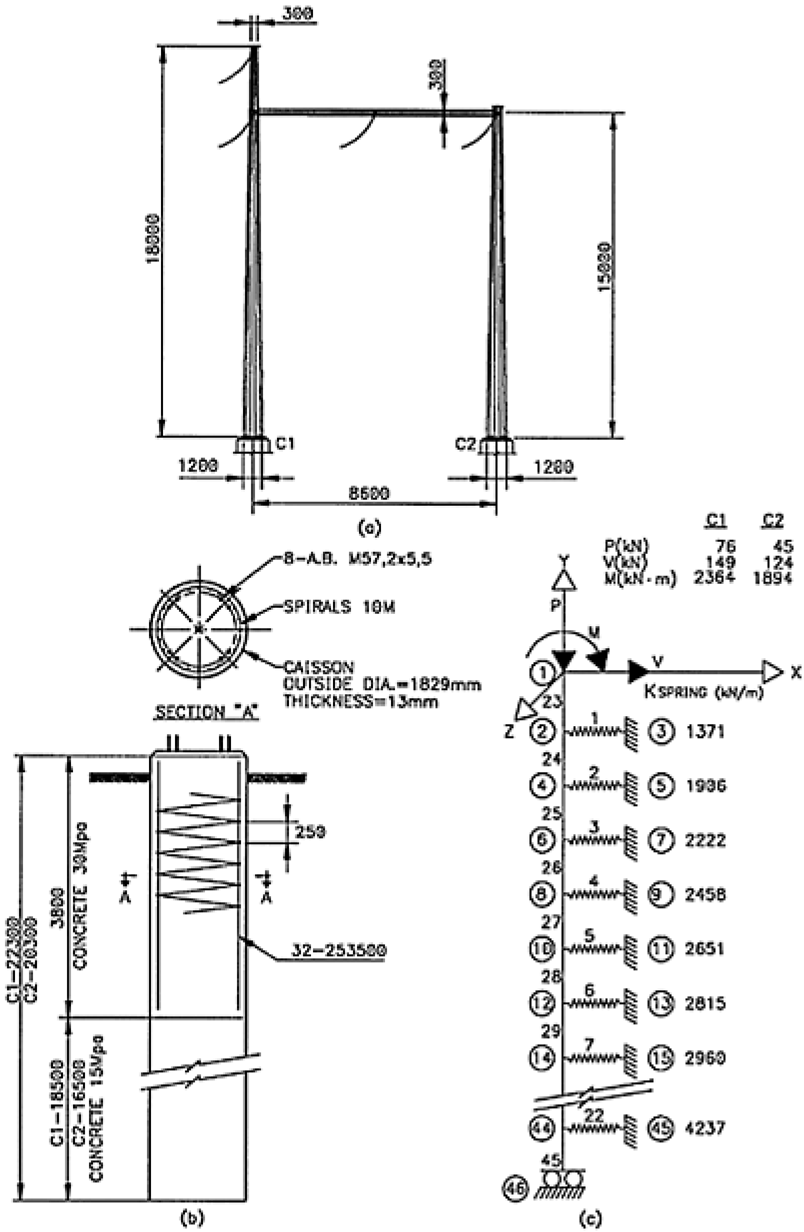

FIGURE 2 H-FRAME PORTAL STRUCTURE WITH CAISSON FOUNDATIONS

or omissions, of course, are the responsability of the author. The author would also like to express his kind appreciation to his superior, Mr Marcel Beauchamp, for his support and encouragement throughtout this work. Also, sincere thanks go to Mr Robert Desrochers for drafting the figures. Financial support for this study was provided by Hydro-Quebec, Postes de répartition ouest, Montréal, Québec, Canada.

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PRELIMINARY DESIGN RULES FOR STRUCTURAL STEEL SHEARHEADS IN CONCRETE SLABS

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ABSTRACT. A structural steel shear-head incorporated within the concrete slab over the support provides an attractive solution for the problem of providing large service holes adjacent to the column. This paper describes the development of an optimum configuration of shearheads and preliminary design rules. Mechanisms for the transfer of load from the slab to the column via the shearhead are described. The main output from the project is to be the publication of a Design Manual for the design and detailing of structural steel shearheads used in reinforced concrete flat slabs. The Design Manual will contain tables of standard prefabricated shearheads with given load capacity and will be published jointly by the British Cement Association, Reinforced Concrete Council and the CRIC (Concrete Research and Innovation Centre), Imperial College.

Keywords: Flat Slab Construction, Structural Steel Shearheads, Design Guidance, Service Holes, Punching Shear.

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INTRODUCTION

A shearhead is a fabricated structural steel assembly placed within the slab depth on the column support. Shearheads have been used widely in the United States and on the Continent for around thirty years but their use has been very limited in the United Kingdom. Shearheads used in the States are generally of a cruciform arrangement with steel members passing through the columns. Generally the top reinforcement passes over the shearhead. The bottom reinforcement may either pass under the shearhead or be curtailed and supported on the bottom flange.

Structural steel shearheads offer many advantages in flat slab construction. The most important of these are:

i) they increase the punching shear resistance,
ii) they give the opportunity for providing large openings close to columns,
iii) they are an efficient and practical method for transferring shear and bending forces to columns especially edge columns,
iv) they increase the maximum column shear capacity,
v) they enable prefabrication of column-shearhead assemblies on large repetitions projects, and
vi) they eliminate the need for using column heads and drops.

Rules for design of the cruciform type of shearhead are given in the ACI Code(1). These are based on work carried out at the Portland Cement Association and reported by Corley and Hawkins(2). Three design criteria are considered:

i) shearheads increase the punching shear perimeter;
ii) the flexural strength of the shearhead arm needs to be adequate;
iii) the slab reinforcement for the negative moments over the columns can be reduced.

In 1989, a project was started at the BCA with support from DTI, Square Grip Ltd and Ove Arups to develop shear reinforcement systems for flat slabs. The first phase of this work led to the development of the Shearhoop system (3,4). The second phase was concerned with structural steel shearheads with the aim of developing a practical system and producing design guidance for UK conditions. This project had to be halted owing to organisational changes at the BCA; recently, the project has been revived at Imperial College following a grant from the DoE and with support from the RCC. This is scheduled for completion early in 1997 with the publication of a Design Manual.

TEST WORK

Extensive test work was carried out to assess the performance of slab reinforced with steel shearheads.

Three series of tests were carried out:

i) Model tests on internal slab-column connections; slabs were 120 mm deep.
ii) Full scale tests on internal slab column connections; slabs were 250 mm deep.
iii)Large scale model tests on edge slab column connections; slabs were 150mm deep.

Details of the test specimens have been published elsewhere[5] and are not repeated here. The results are being analysed in detail to confirm the design rules being proposed.

Shearheads of the type shown in Figure 1 have been developed as the optimum design in terms of structural efficiency. This is prefabricated from channel sections (or I sections for large loads) with the primary arms placed either just outside the column steel or passing through the column section. Welding of these units is to BS 5135 and in accordance with recognised QA procedures.

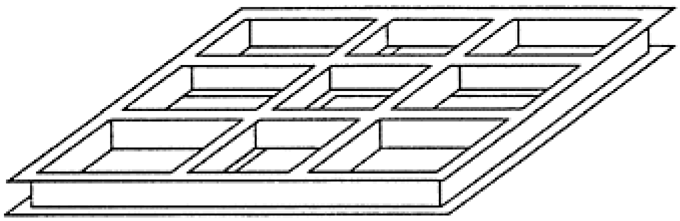

Figure 1 Optimum configuration of steel shearhead

PRELIMINARY DESIGN RULES

This paper is concerned with draft design recommendations for the type of shearhead developed in this project. It should be made clear that these are preliminary rules subject to confirmation with the BCA and other test data currently being analysed. In due course, tables of standard shearheads with given load capacities calculated on the basis of the final approved recommendations will be produced.

The following points should be considered in the design of shearheads:

- location.
- control perimeter.
- plastic moment capacity of steel section.
- provision of openings.
- maximum column face shear.
- maximum shear capacity.
- minimum stiffness of steel section.
- span depth ratio.
- transfer of moment to column.
- reduction in support steel.
- prestressed slabs.

These are discussed in turn.

1. Location

i) Shearheads are positioned within the slab depth so that the top reinforcement passes over the shearhead. The bottom reinforcement may either pass under the shearhead or be curtailed along the line of the bottom flange. Where the bottom reinforcement is curtailed along the line of the bottom flange and the edge of the shearhead lies in the region of sagging moment, the bottom reinforcement needs to be adequately tied to the flange by welding or other means.

In some circumstances, shearheads may be positioned directly on the slab formwork. In such cases, the durability and fire protection aspects must be considered carefully. Fire engineering methods can generally be used to demonstrate adequate fire resistance.

ii) All compression flanges of the steel section shall be located within 0.3d from the compression surface of slab and tension flanges within 0.3d of tension surface of slab.

iii) The primary arms of the shearheads can be located to pass through the columns. Where this interferes with the column steel, it is possible for these arms to be located just outside the column provided the column face shear is not greater than the critical value (see section 6). The channel section can have a secondary plate welded to it so as to form a shear key; it should be noted that the section could be positioned within the cover region of the column.

Small holes are provided within the web of the shearhead to facilitate concrete being provided within the channel and to improve composite action.

2. Control Perimeter

In the ACI standard, enlarged values of the critical perimeter are specified. Extensive test work was carried out at the BCA to investigate this particular aspect of shearhead design. The tests show that, for these shearheads, it will be conservative to take the control perimeter at 1.25d from shearhead face and the v_c values given in BS8110 may be used.

There has to be a maximum limit on the size of the control perimeter. This will be covered under section 5 on maximum shear capacity.

3. Plastic Moment Capacity of Steel Section

The load transfer mechanism of these shearheads is quite different from ACI type shearheads and design rules based on first principles have to be developed for checking the moment capacity of the shearhead arms. The intention is to ensure that the steel section does not fail prior to a punching shear failure outside the shearhead.

The assumption made is that the shear forces along the secondary arms on the perimeter of the shearhead are evenly distributed. The magnitude of this force is $V-V_c'$ where V is the total column shear and V_c' is the product v_c b_o d where b_o is the perimeter of the shearhead. The forces from the secondary arms are then transmitted directly through the primary arms to the columns.

The moment capacity required, M_p, of each primary arm is then given by

$$M_p = 1/n\ (V-V_c')\ l$$

Where 1 is the length of shearhead arm from column face and n is the number of primary shearhead arms.

Generally, the moment capacity of the secondary arms does not have to be checked separately if they are fabricated from the same section. Also the primary steel arms have to be checked for web shear and buckling.

4. Provision of Openings

Openings may be provided anywhere within the shearhead. The sum of the lengths of the openings in any direction should not be greater than one third of the length of shearhead measured in the same direction. The required bending reinforcement interrupted by the openings should be placed either side of the openings up to a maximum distance of d, the effective depth, outside the shearhead.

When openings are provided, the primary arms need to be stiffer because the shear forces carried by the steel section and hence the moment capacity required are increased. V_c' in section (3) above is reduced according to procedures given in CIRIA 110 [(6)] for provision of holes. The calculations for the plastic moment capacity can then proceed as in section (3) above; alternatively it is conservative to take V_c' as zero.

5. Maximum Shear Capacity

This is a problem area as far as the drafting of rules is concerned. The facts are:

i) European literature claims the punching shear resistance can be quadrupled by providing shearheads.
ii) The ACI Code has a limit of 1.75 V_c for shearheads (1.5 V_c for conventional reinforcement).
iii) EC2 refers to 'design by testing' for shearheads. There is, however, a limit on the size of loaded area to 3.5d which is very small as far as the size of shearhead is concerned.

The reason for imposing this limit is the lack of test data on large shearheads. It is suggested that the extension of the primary shearhead arms is limited to 3d from the column face. In practice, this will mean that shearheads are provided within the range 1.5d to 3d from column face. For a 3d extension, the control perimeter is 4c+34d as compared to 4c+12d for a slab without shearhead giving a limit of around 2.5 V_c for the maximum shear force with shearhead. It should be noted that the current limit using normal reinforcement in BS8110 is $2V_c$ and $1.6V_c$ in EC2 ($2V_c$ in NAD).

6. Maximum Column Face Shear

Where the primary arms are placed on the column face, the face shear should be limited to the lesser of $0.8\sqrt{f_{cu}}$ or 5 N/mm^2. Where this limit is exceeded, the primary arms can be located within the column; some of the shear force is then transferred to the column by direct bearing and the remainder along the column face. The maximum shear capacity may be doubled by this means.

7. Minimum Stiffness of Steel Section

The ACI Code requires the ratio α_v between the stiffness of the shearhead arms and that of the surrounding composite cracked slab section of width c= to be greater than 0.15. This requirement is considered unnecessary for these shearheads where the stiffness will be determined from the required plastic moment (see Section 3).

8. Span: Depth Ratios

The BCA test work has shown a significant increase in the stiffness after cracking of the test slab with shearheads (around 50% increase). However, it is likely that in real structures the effects would be less significant owing to continuity of the flat plate beyond the line of contraflexure. A grillage analysis carried out to investigate this further has proved to be inconclusive.

Until further information becomes available, it is recommended that rules for span depth ratios given in BS8110 are used for the design of slabs with shearheads.

9. Transfer of Moment to Column

There is some American test data on ACI type shearheads to suggest that conventional methods for dealing with moment transfer such as in BS8110 are appropriate for the calculations of shear stresses caused by transfer of moments even when shearheads are used. No additional recommendations are necessary.

For edge columns, when unbalanced moments are considered, the shearhead must have anchorage to transfer the moment M_p (see section 3) to the column. This can be done simply by bearing within or outside the edge column.

10. Reduction in Support Steel

The ACI standard allows the support moment for designing the reinforcement to be reduced by the moment carried directly by the shearhead. Some European literature claims that this can be halved in many cases.

There seems no reason why the support moment should not be reduced by the calculated value M_p although in practice this makes little difference to the steel arrangement.

11. Prestressed Slabs

The principles outlined in this note can be applied to prestressed slabs. There may, however, be difficulties in preparing tables of standard shearheads with given load values for prestressed concrete slabs. Further work will be required in this area.

CONCLUDING REMARKS

Preliminary design rules for providing steel shearheads in concrete slabs have been described. Calculations such as those required can be fairly time-consuming and the next phase of this work is to provide a Design Manual with tables of standard shearheads with given load capacities. This Design Manual should be available in early 1997.

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SHEAR RESISTANCE OF REINFORCED CONCRETE MEMBERS AT HIGH TEMPERATURES

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ABSTRACT. An alternative shear reinforcement is proposed in the form of horizontal bars provided at the centre of the cross-section of reinforced concrete beams. A design method is derived for estimating the ultimate shear resistance of beams at normal and high temperatures. The design method accounts for the influence of strength of concrete, tension steel and the reinforcement on the modes of transfer of shear. The design rules are supported by the results of tests on rectangular beams at normal temperature and under exposure to fire. The design rules are also verified using a non-linear finite element analysis.

Keywords: Reinforced concrete beams, Shear resistance, Exposure to fire, Centre bars, Non-linear FE analysis

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INTRODUCTION

Concrete is known to provide good protection to the steel reinforcement, from the environmental effects as well as from exposure to fire. The cover to the steel, therefore, is a main consideration in the design of reinforced concrete members. The prescriptive recommendations for fire rating of structural elements include the cover to the reinforcement and minimum thickness of members. Also, the rules for design against shear require structural elements to have certain minimum dimensions, for example, members with thickness less than 200 mm which cannot be reinforced against shear using links.

In this paper, a design method is developed for estimating the load carrying capacity of members under normal as well as fire exposure conditions, using the centre bars as shear reinforcement. This method could allow flexibility of choice of dimensions of members and the strength and properties of the constituents of the members.

WHY CENTRE BARS ?

Centre bars would serve as shear reinforcement for members irrespective of their size. Also, their contribution could be available for a longer period of exposure to fire, due to the protection provided by the surrounding concrete. The centre bars could assist in reducing the requirement of links and improving the detailing of steel at supports by avoiding congestion of steel. In addition to the extra shear carrying capacity, members provided with centre bars could have an effective tying and an additional ductility which is important for design against accidental loading and for providing an improved residual resistance.

SHEAR RESISTANCE OF REINFORCED CONCRETE MEMBERS AT HIGH TEM PERATURES

The design of reinforced concrete members against shear failure mainly concerns provision for avoiding a brittle and sudden failure. Although there is no conclusive solution available to the problem of shear, some general principles can be deduced from the research by Kani [1], Regan [2] and Taylor [3]. A reinforced concrete member is able to carry the applied shear through three modes of shear transfer; reactive stresses in the compression block concrete, aggregate interlock across a shear crack and the dowel action of the tension steel. Also, the reduction in the depth of compression block is considered to be the main damaging effect of the increasing shear force.

These shear transfer modes are influenced by the components of the section; concrete, the tension steel and the web steel. Their influence changes with the increase in applied shear. However, the increase and decrease in the individual contributions of the components could compensate each other. For example, the aggregate interlock provided by the concrete could be partly taken over by links. The links would assist in bridging over the local weakness in concrete section and redistribute the stresses and also enhance the resistance to dowel-splitting in the tension steel region. The tension steel provides the stiffness which enables the member to resist the worsening of cracks. This depends on the amount of tension steel and its modulus of elasticity. These contributions of the constituents of the member are combined in a rule to give a part of the ultimate shear resistance of concrete (V_{cu}). V_{cu} depends mainly on the strength of concrete and, also, on the amount and modulus of elasticity (E_{st}) of the tension steel.

$$V_{cu}=0.0046\ (\rho\ E_{st}f_{cu})^{1/3}\ (400/d)^{0.25}\ bd \quad (1)$$

b	width of beam	(mm)
d	effective depth of the beam (measured from the extreme compression fibre to the centroid of tension steel)	(mm)
fcu	Characteristic cube strength of concrete	(N/mm^2)
ρ	$100A_{st}/bd$, where A_{st} is the amount of tension steel	

With the value of E_{st} of 200 kN/mm^2, this rule gives the corresponding BS 8110 rule. The links have a complex role in enhancing the shear carrying capacity of a section.

Ideally, the links should be treated as reinforcement acting in conjunction with the concrete and enhancing the shear resistance of a member. For the sake of convenience, the ultimate shear resistance (V_{DU}) of a member reinforced with links is expressed as addition of two components; one provided by concrete, V_{CU} given by the previous equation and the other provided by links, V_{LU}.

$$V_{LU}=(A_{sw}f_{yv}d)/s \quad (2)$$

f_{yv}	yield stress for steel reinforcement	(N/mm^2)
A_{sw}	area of cross-section of links	(mm^2)
S	spacing of links along the length of the member	(mm)

Shear Resistance of Beams with Centre Bars

Under this project, it was decided to look beyond the normal method of providing links for avoiding shear failure. Hence, the design rules have been developed for an alternative form of shear reinforcement as horizontal bars at the centre of the section. The ultimate stage contribution of the central steel, V_{BU}, is obtained on the basis of results of tests on beams, with measurement of stresses in links and the bars using strain gauges. The rules are supported analytically and they have been verified using ABAQUS computer program.

$$V_{BU}=0.4\rho_b V_{CU}\leq 0.4\ V_{CU} \quad (3)$$

ρ_b $100A_b/bd$ (A_b=area of cross-section of central horizontal bar, mm^2)

Similar rules are developed for flat slabs. The central bars will be specially useful for slabs with depth less than 200 mm which cannot be reinforced with links.

The ultimate shear resistance (V_{DU}) is given by the following rule :

$$\mathbf{V_{DU}=V_{CU}+V_{LU}+V_{BU}} \tag{4}$$

Shear Resistance of Beams at Elevated Temperatures

The rules have been adapted for evaluating shear resistance of beams under elevated temperature conditions. In this way, the estimates of shear resistance account for the change in strength and properties of concrete, the tension steel and the web reinforcement under fire exposure conditions.

$$V_{CT} = 0.0046\left(\frac{100\ A_{ST}}{b_T d_T} E_T f_{cT}\right)^{1/3} \left(\frac{400}{d_T}\right)^{0.25} \frac{b_T d_T}{1000} \tag{5}$$

$$V_{LT} = \frac{A_s d_T f_{yT}}{s} \tag{6}$$

$$V_{BT} = 0.4\ (V_{cT})\ \left[\frac{f_{cTm}}{f_{cT}}\right]^{1/3} \frac{100 A_b}{b_T} d_T \quad kN \tag{7}$$

In these expressions, the suffix T is used to show the following parameters :

i) the contributions of concrete, links and the central bar at T°C;
ii) the reduced dimensions (b_T and d_T) ignoring area of section where the temperature is in excess of 750°C; and
iii) the reduced strengths of concrete and steel at T°C.

f_{cT} is the average strength of the concrete section, but f_{cTm} is the concrete strength at the location of the central bar.

The reductions in strengths are carried out according to the current draft of Part 1.2 of Eurocode EC2. In particular, the term ET denotes the reduced modulus of elasticity of the tension steel., which signifies the loss of stiffness causing reduction in the resistance to worsening of cracks.

The application of these rules requires estimates of temperatures developed within the cross-section of the beam after a certain fire exposure period. The rise in temperature in a concrete section as a response to the external high temperatures depends on a large number of factors. These factors include the moisture content in the concrete and the chemical composition of the aggregate and cement. Also, the temperature development in a beam depends on the heating conditions and heat transfer characteristics of the environment. However, these factors cannot be conveniently evaluated for the purposes of developing a general design rule. Therefore, the information is based mainly on the data derived from tests; for example, the graphs prepared by Wade [4] for beams exposed to fire on three sides. These graphs and the temperature measurements obtained by Lin [5] are used for constructing mathematical equations to give temperature profiles in beams.

The temperature (T°C) developed at a point located at a distance of “x” mm from the face of the beam is assumed to be governed by the following factors:

i) the ambient high temperature which is a function of the fire exposure time (t, in minutes);
ii) b, the width of the cross-section (mm); and
iii) r, the ratio of the overall height to the width of the beam.

The following cubic equation is proposed and the values of D, A, B, and C are obtained by solving a number of simultaneous equations constructed to represent, as closely as possible, the trends given in Wade's charts and to accord with measurements by Lin and the data obtained from the tests done under this project.

$$T=(D-Ax+Bx^2-Cx^3)/r^{0.25} \tag{8}$$

in which, $D=475\ r^{7/12}-(b-105\ t^{1/3})$

$A=3.33\ (3+0.0033t+(100-t)/b)$

$B=0.085$ $C=0.000221$

These equations can be used to plot Graphs, to give temperature distribution in beams with various widths and for various fire exposure periods. Using these equations, the formulae for $\mathbf{V_{DU}}$, $\mathbf{V_{CT}}$, $\mathbf{V_{LT}}$ and $\mathbf{V_{BT}}$ are set in a LOTUS Spreadsheet program which is easy to operate.

Beam Tests

Ten rectangular simply supported beams were tested at Veseli, near Prague under fire exposure conditions. The soffit and two vertical faces of the beams were exposed to fire conforming to the standard time-temperature relationship. During the tests, failure modes of the beams were noted and mid-span deflections and temperatures developed in the beam were recorded. The beams were 200 mm wide and 300 mm deep and the span was 1400 mm. The cover to main steel was 25 mm. The other details of the specimens are shown in Table 1 and Figure 1.

All the beams failed in shear and the ratio of W_{est}/W_t is close to 1.00 for all beams except beams B101 and B402. The specimen B101 was used as a trial specimen and the maximum applied load was 56 kN. The trial test was useful in taking some additional measures for the other tests; for example, additional insulation was provided on the top of the beam for the subsequent tests to protect it from heat from the furnace. Beam B402 failed earlier than expected and the temperatures developed

Table 1 Details of beam specimens

Spec No	f_{cu} N/mm^2	Top steel	Tension steel	Centre steel	Links
B101	39.6	2T12	3T20	–	–
B102	39.5				
B301	34.4	”	”	1T16	–
B302	34.0				

B401	42.9	”	”	1T20	–
B402	41.5				
C101	42.3	”	”	–	T6@200
C102	42.9				
D201	38.5	”	”	1T16	T6@200
D202	38.4				

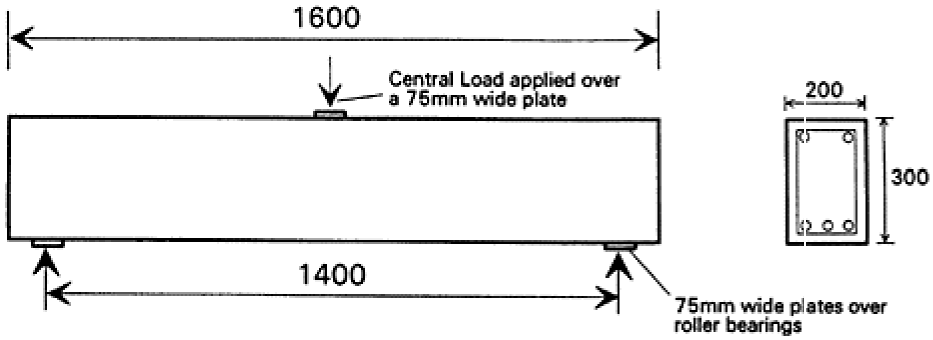

Figure 1 Detail s of test beams

in the beam at failure were much lower than the other beams. This could be attributable to some local weakness resulting in severe spalling near the support, which was noted during the test. The spalling could have caused direct exposure of tension steel to fire and a rapid reduction in the strength and modulus of elasticity of the steel.

FINITE ELEMENT ANALYSIS

A non-linear finite element method was used to analyse the beams in order to verify the contribution to the shear resistance capacity made by the provision of centre bars. The software used was ABAQUS. Due to symmetry only half the beam was idealised using, C3D20R, three-dimensional brick type elements with 20 noded reduced integration. The REBAR facility available in ABAQUS allowed reinforcing bars to The test results are compared with the estimated load carrying capacity as shown in Table 2.

W_t : The centrally applied load (kN). W_t is twice the applied shear, calculated as 60% of the load carrying capacity of the beam at room temperature using a notional value of f_{cu} of 30 N/mm^2.

W_{est} Estimated load-carrying capacity corresponding to the duration of the test (t_t, minutes) [twice the value of the estimated shear V_{DT}];

δ_m: The mid-span deflection (mm)

Table 2 Estimates of load-carrying capacity compared with test results

Spec no	tt minutes	W_t kN	W_{est} kN	W_{est}/W_t	δ_m mm
B101	126	56*	70	1.25	10
B102	113	70	76	1.08	10
B301	128	80	78	0.98	12
B302	101	”	92	1.15	12
B401	102	90	107	1.19	12
B402	56	”	128	1.42	6
C101	111	110	97	0.88	7
C102	101	it	105	0.95	12
D201	107	120	112	0.93	11
D202	94	”	122	1.02	11

be specified anywhere in the solid elements. The reinforcing bars were defined individually at the specified locations in the cross-section. The details of the beam are shown in Figure 2. For this study three of the beams containing the centre bars are discussed. These are B300, B400 and D200. Beams B300 and B400 contained 1–16 mm $\phi$and 1–20 mm ϕcentre bars and beam D200 contained 1–16 mm ϕcentre bar plus 6 mm ϕvertical links @ 200 mm centres (Table 1).

The material non-linear properties adopted for concrete is defined by a uniaxial stress-strain relationship as shown in Figure 3. The peak stress of 0.67 fcu represents the maximum stress in concrete under uniaxial stress condition. For the reinforcing bars a bi-linear stress-strain relationship, Figure 4, was used to define its uniaxial behaviour in the elastic and plastic regions.

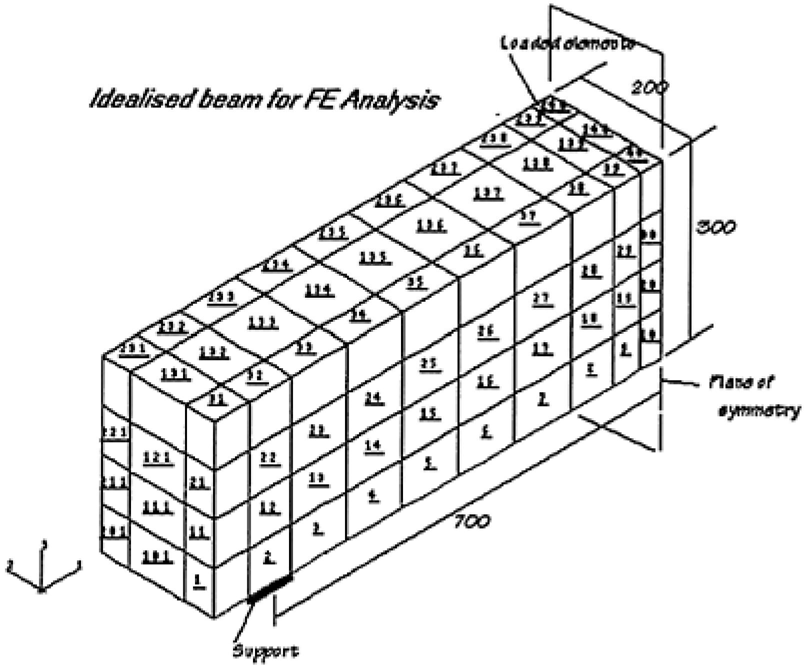

Figure 2 Idealised beam for ABAQUS analysis

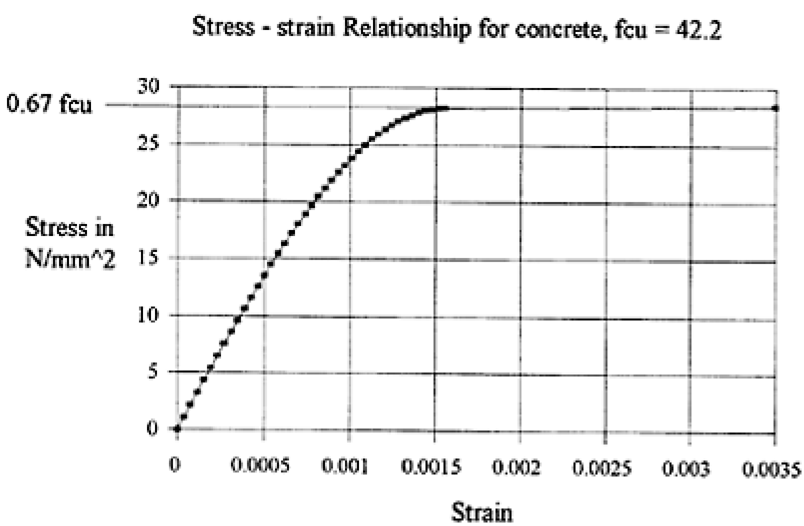

Figure 3 Stress—strain relationship for concrete in compression

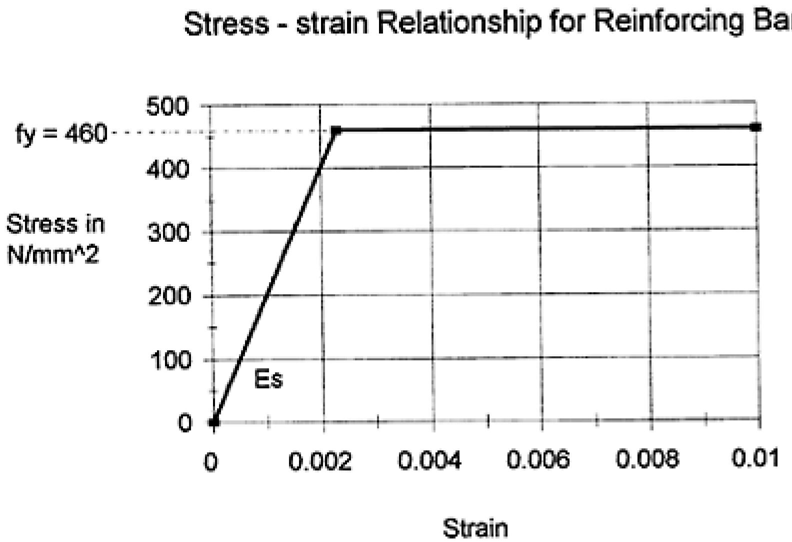

Figure 4 Stress—strain relationship for reinforcement

The FAILURE RATIOS parameters define the ratios of concrete strength in tension and in multi-axial stresses. The TENSION STIFFENING curve defines the softening of the concrete behaviour after the formation of cracks. The SHEAR RETENTION parameters define the deterioration of shear stiffness in terms of concrete strain after the event of cracking. The parameters defining the failure ratios, tension stiffening and shear retention are important parameters which control the analysis process of concrete beams. For this study the parameters were selected as follows :

*FAILURE RATIOS
1.16, 0.12, 1.28, 0.33
*TENSION STIFFENING
1.0, 0.0
0.0, 3.5 E-3 (in some cases 3.55×10^{-3} was used)
*SHEAR RETENTION
1.0, 0.0075, 1.0, 0.0075

The details with regard to data preparation and input file may be fond in ABAQUS User Manual [6].

FE ANALYSIS RESULTS

The results from the finite element analysis and the ultimate shear capacity of the beams calculated from the equations (1) to (4) are compared in Figures 5 and 6. In Figure 5 beams B300 and B400 series are analysed for centre bars $16\,\phi, 20\,\phi, 25\,\phi$ and 32ϕ representing 0.38, 0.59, 0.92, and 1.51 percents respectively. In Figure 6 D200 series beams (with links) are compared.

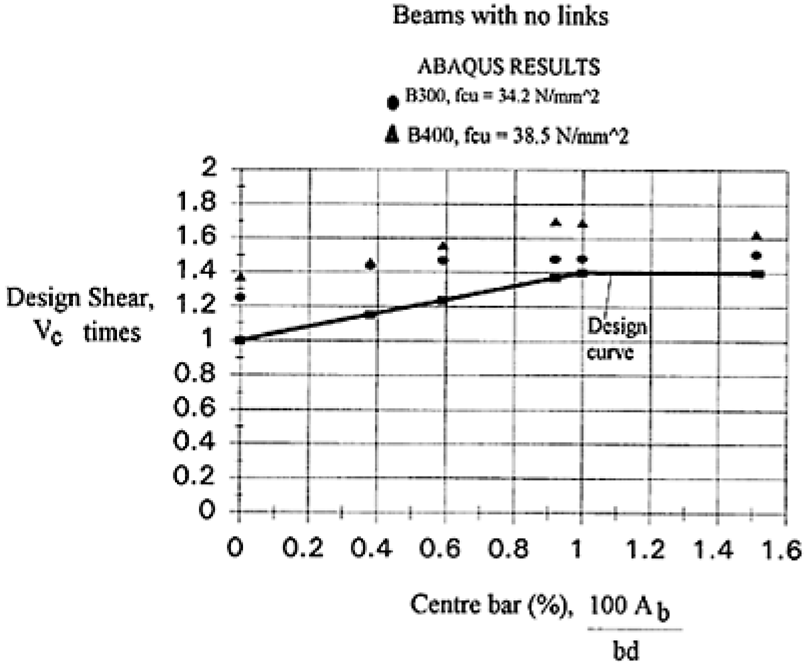

Figure 5 FE analysis vs design: Beams with no links

In both figures, 5 and 6, the slopes of the curves represent the proposed contribution of the centre bar from zero up to a maximum of 40% of V_c for 1% reinforcement. In Figure 6 the contribution of links is about 73% of V_{cu} for series D200 beam. In all three series of beams studied here the FE analysis confirms that the proposed design equations are good and agree well with the analysis. From the FE analysis, although it is not possible to extract exactly the amount of contribution made by the centre bars, the overall estimate of shear resistance capacity are in agreement with those obtained from the design rule.

CONCLUSIONS

1. The centre bars provide enhanced shear resistance of reinforced concrete beams at normal as well as at high temperatures.
2. A method of design based on simple equations is proposed. The method provides an estimate of the ultimate shear resistance capacity of beams taking into account the contribution of the centre bars.
3. A non-linear finite element analysis confirms the adequacy of the estimate of the overall capacities of beams given by the proposed design method.

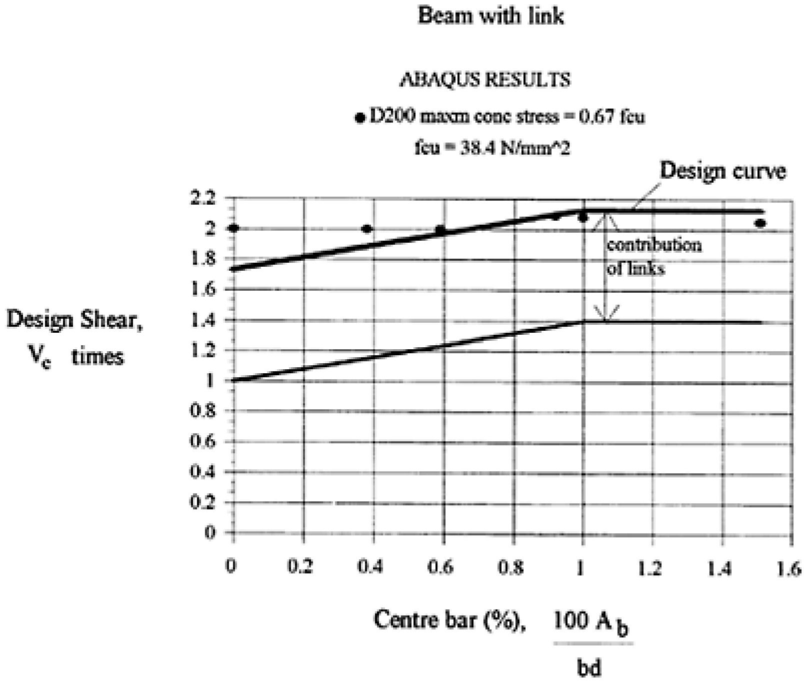

Figure 6 FE analysis vs design: Beam with links

4. The method is extended for evaluating shear resistance of beams exposed to fire. Tests carried out on a series of beams confirm that the proposed method of analysis for predicting the shear resistance of beams with centre bars.

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REPEATED AND CYCLIC LOADING TESTS ON PRECAST CONCRETE BEAM-TO-COLUMN CONNECTIONS

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ABSTRACT. This paper describes an experimental investigation into the behaviour of two types of reinforced concrete beam-to-column connections. Referred to as Types A and B, they are recommended by the Precast/Prestressed Concrete Institute and the Australian Prestressed Concrete Group for use in precast building frames. A total of 12 half-scale interior connection models were designed, built and tested to failure to evaluate their strength and ductility properties under unidirectional repeated and cyclic loading. For comparison, 12 additional models were investigated under static loading conditions. A summary of the findings from the repeated load tests is presented together with a more detailed discussion on the tests under cyclic loading. It is found that the bending strengths of the precast connections are higher than the monolithic connections. In addition, their ductility and energy absorbing capacities are superior to their monolithic counterparts.

Keywords: Beam-to-column connections, Cyclic load, Deformation, Energy absorption, Model tests, Precast concrete, Reinforced concrete, Repeated load, Strength.

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INTRODUCTION

Connection design is one of the most important considerations for the successful construction of precast reinforced concrete structures. The detailing and structural behaviour of the connection affect the strength, stability, constructibility as well as load redistribution of the building under loads.

Despite the fact that precast concrete connections have been used all over the world since the 1950's, a very limited number of studies has been conducted on the performance of precast concrete connections. Although the PCI manuals [1,2] contain the descriptions of approximately 40 beam-to-column connections fulfilling many functions, published test results are available for only a few of them. Reliable connection behaviour can only be assessed by physical testing.

A laboratory study has been conducted of the strength and deformation behaviour of beam-to-column connections suitable for use in precast reinforced concrete building frames. In all, 24 half-scale model connections were designed, built and tested. They include,

- for the unidirectional repeated and cyclic load tests, 2 models each of the monolithic and precast connection Types A and B [3,4];
- for the additional static load tests, 4 monolithic models and 4 each of the precast connection Types [3,5].

The behaviour of the connections under static and repeated loads has been reported elsewhere [6]. Thus only a summary of the findings is given here for completeness. This paper presents the results in detail of the cylic load tests on the precast connections.

TEST MODELS

The design of the models was based on the structural requirements of a five-storey reinforced concrete frame forming part of a low-cost residential building system [7]. The two types of precast connections were designed according to recommended guidelines [1,2,8,9]. The reinforced concrete design and manufacturing process complied with the Australian Standard [10,11].

In all, 24 half-scale connection models were fabricated, making eight groups of two precast specimens (Types A and B) and one monolithic specimen. All models had the same overall dimensions but different groups had different concrete strengths and/or steel ratios. Of the models, twelve were tested under unidirectional repeated and cyclic loading. The remaining twelve were investigated under static loading. Each model was identified by two letters and a number. The first letter, C, R or S, indicates cyclic, repeated or static loading. The second letter, M, A or B represents monolithic construction or precast connection Type A or B. The number at the end identifies the different tensile steel contents.

About 0.1m^3 of concrete was required to cast each model. Commercial pre-mixed concrete was used in the model construction. Cast-in-place concrete was mixed in the laboratory when assembling the components. The structural details of precast connection Types A and B are given in Figs. 1 and 2 respectively. Fig. 3 together with Table 1 summarises the material properties and the cross sectional details of the connecting beams and columns of the models.

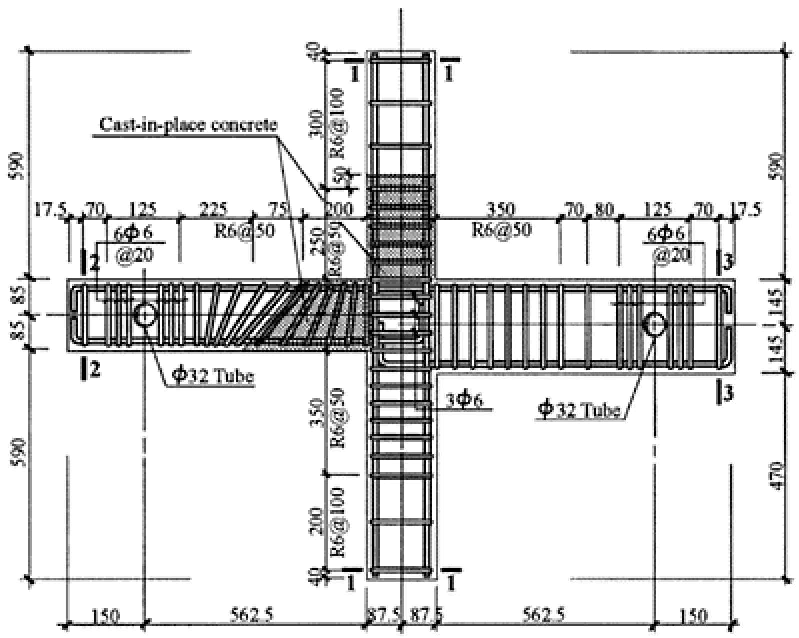


Figure 1 Precast connection Type A

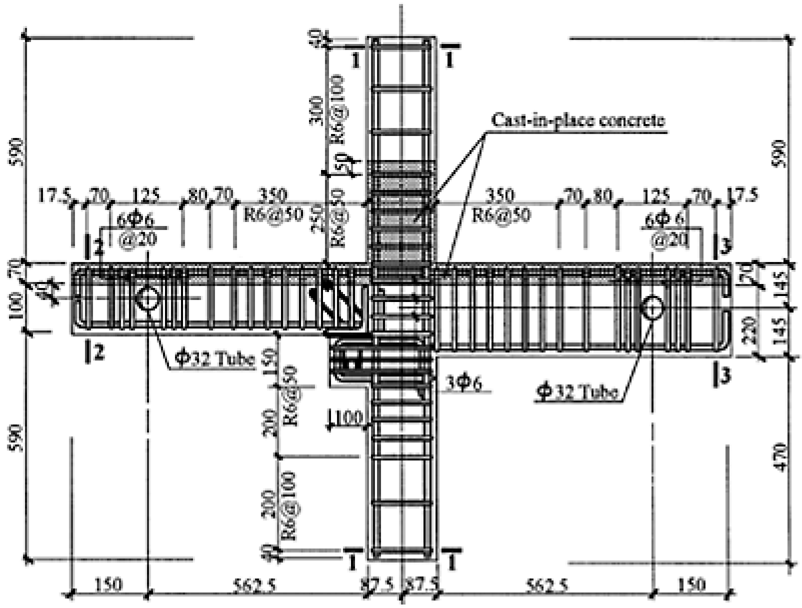


Figure 2 Precast connection Type B

Table 1 Details of beams and columns (refer to Figure 3)

		Reinforcements* of Type of Connecting and Frame Beams					Reinforcements* of Columns			Precast Concrete Strength	Cast-inplace Concrete Strength
Groups	Name of Specimens	Connections	Top		Bottom						
			Area	f_{sy}	Area	f_{sy}	Cover	Area	f_{sy}		
			(mm^2)	(MPa)	(mm^2)	(MPa)	(mm)	(mm^2)	(MPa)	(MPa)	(MPa)
	SM1	Mono.†	400	440	160	372	25	440	440	30	
1	SA1	A	400	440	160	372	25	440	440	30	59
	SB1	B	400	440	160	372	25	440	440	30	59
	SM2	Mono.	600	440	160	372	25	440	440	30	
2	SA2	A	600	440	160	372	25	440	440	30	59
	SB2	B	600	440	160	372	25	440	440	30	59
	SM3	Mono.	330	440	160	325	27	440	440	13	
3	SA3	A	330	440	160	325	27	440	440	13	67
	SB3	B	330	440	160	325	27	440	440	13	65
	SM4	Mono.	330	440	160	325	27	440	440	53	
4	SA4	A	330	440	160	325	27	440	440	53	78

	SB4	B	330	440	160	325	27	440	440	37	60
	RM1	Mono.	400	440	160	372	25	440	440	37	
5	RA1	A	400	440	160	372	25	440	440	37	67
	RB1	B	400	440	160	372	25	440	440	37	67
	RM2	Mono.	600	440	160	372	25	440	440	37	
6	RA2	A	600	440	160	372	25	440	440	37	67
	RB2	B	600	440	160	372	25	440	440	37	67
	CM1	Mono.	330	396	220	396	25	440	396	30	
7	CA1	A	330	396	220	396	25	440	396	32	66
	CB1	B	330	396	220	396	25	440	396	32	66
	CM2	Mono.	600	392	440	392	25	880	392	32	
8	CA2	A	600	392	440	392	25	880	392	34	71
	CB2	B	600	392	440	392	25	880	392	34	71

*For tie spacings see Figure 3

†Monolithic

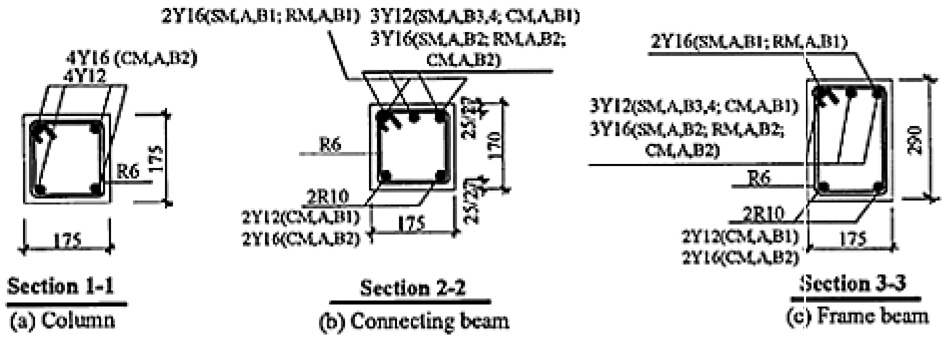


Figure 3 Cross sectional details of of beams and columns (refer to Figures 1 and 2 and Table 1)

TEST SET-UP AND EXPERIMENTAL PROCEDURE

The loading apparatus consisted of three floor-mounted steel portal frames, as shown in Fig.4. The required loads were applied by the hydraulic jacks. The top end of the column was fixed to the cap, which was fixed to load frame 1 through jack 1. The free end of the connecting (precast) beam was loaded by the vertical double-acting actuator or jack 2 (INNERPAC RAH-306) with jack 3 providing the balance for jack 2 at load frame 3. The loads were measured using “Interface” load cells (model 1220-BF with 113.5 kN capacity). The vertical deflection of the connecting beam directly under the loading point was measured by dial gauge 1. The beam and column deflections and concrete strains were recorded manually up to failure. The load-deflection curves of the models under repeated and cyclic loading were drawn with the aid of a Hewlett Packard plotter. The strains on the reinforcing bars at the connecting zone were measured using 10 mm electrical resistance strain gauges. The strain values were recorded using a Hewlett Packard 3054A automatic Data Acquisition/Control System.

For every model test, an axial load (P_c) was first applied on the top of the column. This load, which was equal to 10% of the design axial strength of the column, was kept constant throughout the test. Then a vertical load, P_b, was applied to the connecting beam stage by stage until failure of the model occurred. For the repeated and cyclic loading test, the load P_b was controlled by the magnitude of the vertical deflection, A, measured at the tip of the beam (dial gauge 1, Fig. 4). The vertical tip deflection was increased in multiples of Δ_y, where Δ_y is the deflection at first yield. Typical load history diagrams for the repeated and cyclic loading tests are shown in Figs. 5 and 6 respectively.

In between load applications, visual inspection and manual marking of cracks and crack propagation were carried out. Failure was indicated by a marked increase in beam deflection accompanied by a.rapid decrease in the vertical load P_b.

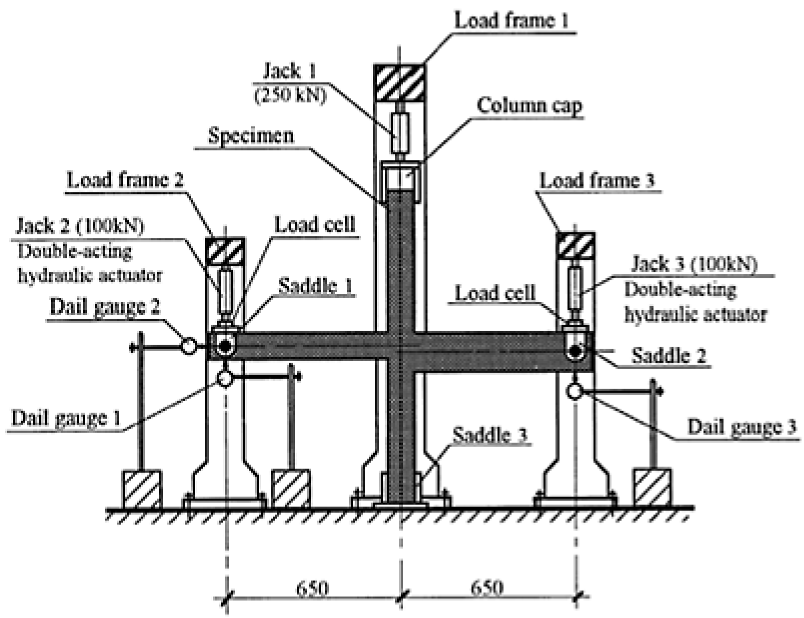


Figure 4 Test set-up

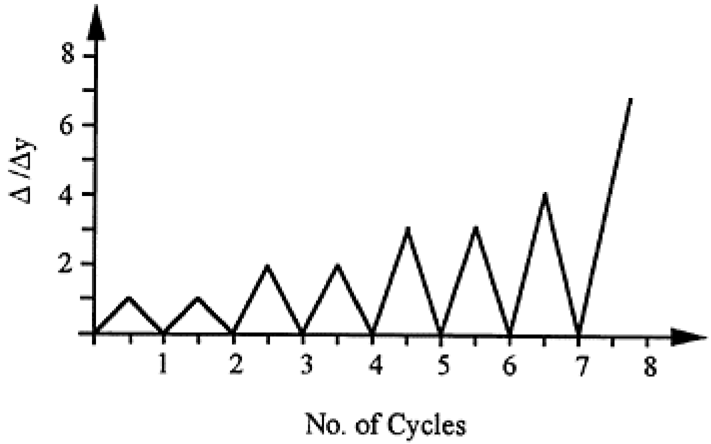


Figure 5 Load history for repeated tests

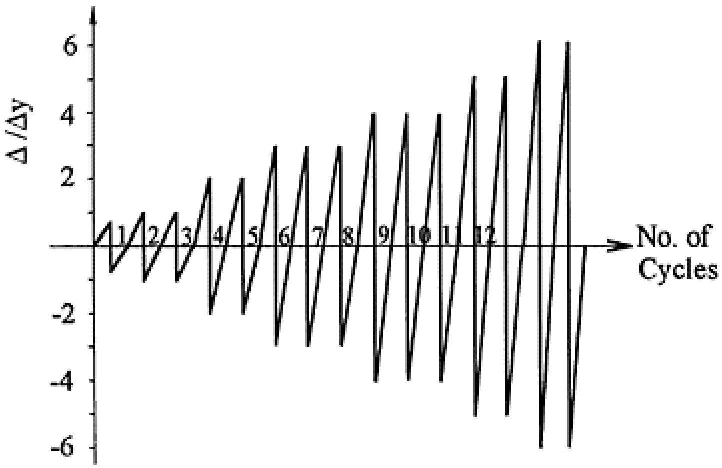


Figure 6 Load history for cyclic tests

STATIC AND REPEATED LOAD TEST RESULTS -A SUMMARY

Based on the model test results, the following observations were made [6]:

- Under both static and repeated loading, the precast connections attained a higher flexural strength than monolithic connections.
- Under static loading, the ductility performance of Type B precast models is satisfactory when compared with that of the monlithic connections. In this respect, Type A connections are superior to Type B connections and the monolithic models.

- Under repeated loading, the ductility characteristics of both types of precast connections are satisfactory, although Type B connections performed marginally better than Type A connections.
- Both the precast connection types, under repeated loading, possessed larger energy-absorption capacities than the monolithic models.

CYCLIC LOAD TEST RESULTS AND DISCUSSION

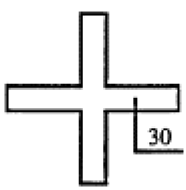
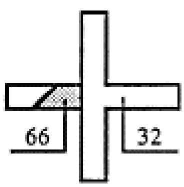
Flexural Strength

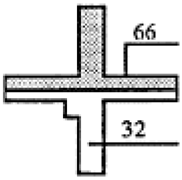
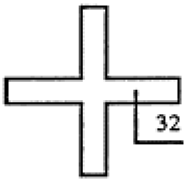
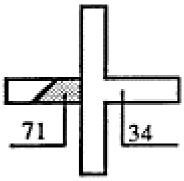
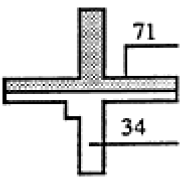
The measured and predicted ultimate loads of the connecting beam are presented in Table 2. Similar to the repeated load tests, it was found that P_u of the precast models under cyclic loading were greater than those of their monolithic counterparts (with the exception of model CA1). The improved performance was due mainly to the strength of the cast-in-place concrete which was much greater than the concrete strength of the component beams and columns and the corresponding monolithic models (see Table 2, column 2).

Deformation and Ductility

The ductility factors, Δ_u/Δ_y, of all the connecting beams are given in Table 2. By comparing models CM2, CA2 and CB2, it is clear that both types of precast connections exhibited good ductility behaviour without strength degradation. For the

Table 2 Test results under cyclic loading

Name	F_c' (MPa)	P_y (kN)	P_u (kN)	P_{max} (kN)	$\frac{P_u}{P_{max}}$	Δ_y (mm)	Δ_u (mm)	$\frac{\Delta_u}{\Delta_y}$	Energy Absorption (kNmm)
CM1		20	31	24.4	1.27	3.97	7.82	2	183
CA1		20	29	31.1	0.93	3.23	16.2	5	1499

CB1		25	43.5	34.7	1.25	4.31	12.9	3	720
CM2		28	52	48.2	1.08	4.85	14.6	3	484
CA2		37	67	52	1.29	5	30	6	4907
CB2		39.1	69	58.8	1.17	5.9	17.7	3	1630

models with a lower tensile steel content (Group 7), the precast connections performed better in ductility than the monolithic one.

A typical load-deflection curve for the group 7 models under cyclic loading are given in Fig. 7. From all of these curves (for groups 7 and 8 models) [4], the following observations were made:

• The load-deflection curves of the precast models are very similar to those of the monolithic models. This indicates that the deformation behaviour of the precast connections is similar to that of the monolithic ones.
• The precast models sustained more cycles than their monolithic counterparts. They generally experienced no more degradation in load carrying capacity than the monolithic ones.
• The Type B connections showed a higher load carrying capacity and smaller crack width at failure. They performed marginally better than Type A.

Energy Absorption

For all the models, the values of the cumulative energy absorbed are calculated and presented in Table 2. It is clear that both types of precast models performed better than the monolithic counterparts in absorbing energy. In addition, Type A models were superior to Type B in this respect.

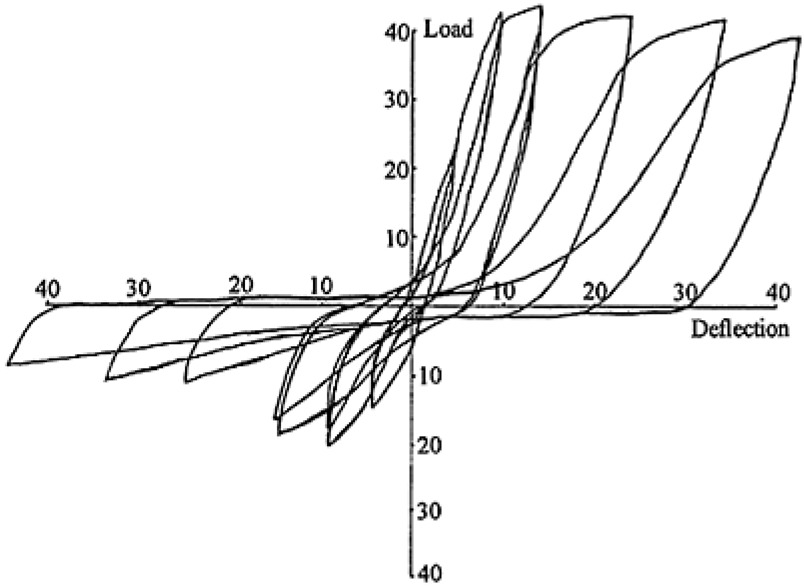


Figure 7 Load-deflection curve for CB1

CONCLUSIONS

Based on the test results of the half-scale beam-column connection models, the following conclusions can be drawn.

(1) Under cyclic, repeated or static loading, the precast connections, in general, attained a higher flexural strength than the monolithic ones.
(2) Under cyclic loading, the precast models were capable of sustaining large beam rotation and deflection. Hence the two types of precast connections are considerd to have performed satisfactorily.
(3) Both the precast connection types, under cyclic loading, possessed a larger energyabsorbing capacities than the monolithic models.

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SISMO-BUILDING TECHNOLOGY: PLAIN CONCRETE IN HIGH RISE BUILDINGS

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ABSTRACT. A theoretical study is presented, based on the Dutch code NEN 6720 [1], to define which bending moment a plain concrete SISMO® wall can resist under a given normal force, allowing a certain tensile strength in the concrete. With these results the outer walls of high rise buildings, designed as a braced structure, were calculated. It showed that plain concrete SISMO® walls can fulfil all demands of stability and serviceability. Also an experimental program was started to define the characteristics of the SISMO® standard solutions for reinforcement. Both theoretical and experimental results are being combined to design the detailing of the bracing and other specific parts of the structure. The great potential of flexibility using SISMO® Building Systems facilitates design and allows modifications to enable standard solutions for reinforcement, if plain concrete is not satisfactory.

Keywords: Building technology, Structural design.

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INTRODUCTION

SISMO® is a modular building system. Each module consists of a 3D-frame of galvanized steelwire (Ø 2,2 mm).

The outer sides of the frame are closed by means of strips, e.g. expanded polystyrene, used as a stay-in-place form as shown in Figure 1.

The structure of each module is variable, depending on the design. The system is developed to combine the advantages of prefabricated, industrial production with the flexibility of on the site production. Different standard SISMO® types are available. Each type is defined by the capital "S", followed by the total thickness of the module in mm. For example "S200" refers to a SISMO® module with an overall thickness of the lattice equal to 200 mm, leaving for the structural material (concrete) a thickness varying between 100 mm and 180 mm depending on the choice of the outer infill materials (at both sides!).

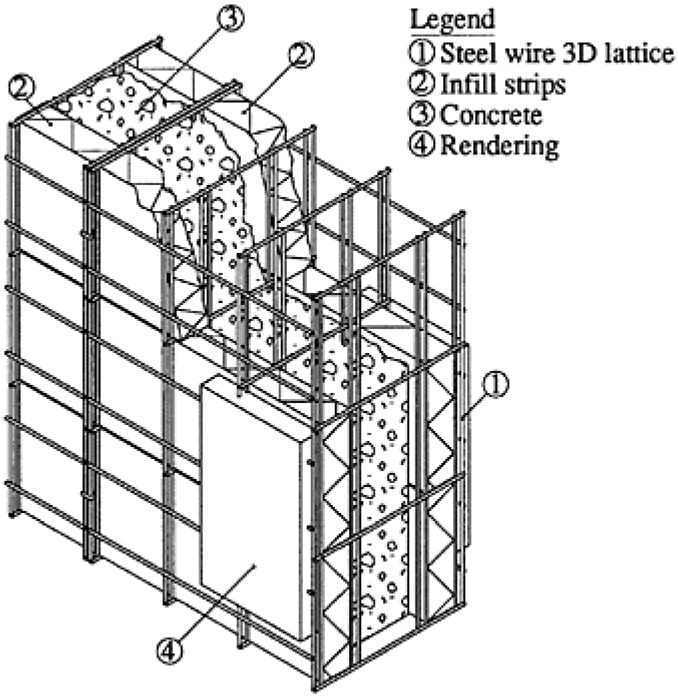

Figure 1. 3D-view of a SISMO® element

BASIC PRINCIPLES FOR DESIGNING PLAIN CONCRETE

The theoretical study of plain concrete is based on the Dutch code NEN 6720 (1991) [1]. It is however obvious that any other code can be adopted, provided that it is sufficiently developed with regard to plain concrete design. The stragegy is to exploit concrete to the ultimate and to use standard solutions for traditional reinforcement where needed. The optimal result is gained when SISMO® walls are designed as braced construction elements whose horizontal loads are supported by other, bracing elements belonging to the same construction, e.g. shear walls. In that case, the outer walls of a (high rise) building are subjected to axial load with a given excentricity, and windload perpendicularly to the plane of the wall. According to NEN 6720, such walls can be designed in plain concrete, provided that $m_d+m_{t.i.}\leq m_u$ (1), where

m_d design value for the limit state of collapse, of the maximum bending moment per unit length, due to the loads liable to act on the structure (Nmm/mm)

m_u ultimate bending moment per unit length occurring with the design value of the axial load applied at the center of gravity of the cross section (Nmm/mm)

$m_{t.i.}$ design value for the limit state of collapse, of the accidental restraint moment per unit length (⅓ of the adjacent moment of span for beams and one-way slabs and ½ of the adjacent moment of span for two-way slabs, according to NEN 6720, § 7.3.3.).

In most cases $m_{t.i.}$ is too high to meet condition (1). Therefore, the Dutch code allows a lower value if this can be proved by applied mechanics, e.g. by proving that a position equilibrium is possible for the limit state of serviceability.

The Axial Force-Bending Moment Interaction Diagrams

In order to calculate the ultimate bending moment for plain concrete under a given axial force, two interaction diagrams were set up: one for the limit state of collapse $\left(n'_d - m_u\right)$ and one for the limit state of serviceability $\left(n' - m_{rep}\right)$ where

n'_d design value for the limit state of collapse, of the axial load p.u. length due to loads liable to act on the structure (N/mm)

n' design value for the limit state of serviceability, of the axial load p.u. length due to loads' liable to act on the structure (N/mm)

m_{rep} bending moment in the limit state of serviceability

For both limit states the stress-strain curves for concrete according to NEN 6720 are given in Figure 2.

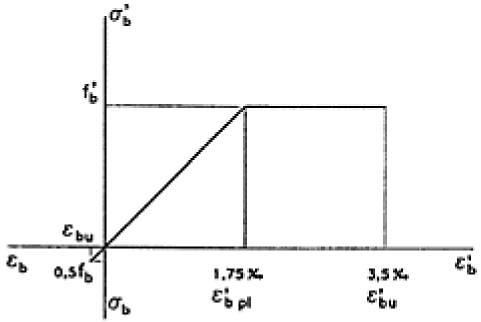

Figure 2a. Stress—strain curve for concrete with regard to the limit state of collapse (NEN 6720)

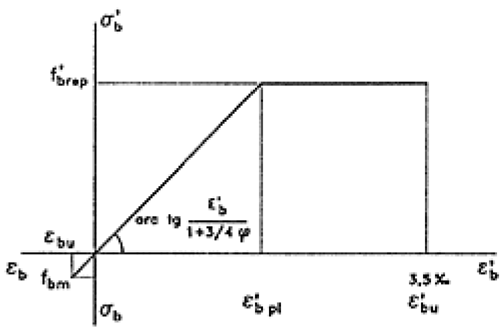

Figure 2b. Stress—strain curve for concrete with regard to the limit state of serviceability (NEN 6720)

Where σ'_b compressive stress in concrete
σ_b tensile stress in concrete
ε'_b compressive strain in concrete
ε_b tensile strain in concrete
f'_b design value of concrete compressive cube strength
f'_{brep} representative value of concrete compressive cube strength
$\mathbf{f_b}$ design value of concrete tensile strength
$\mathbf{f_{bm}}$ average value of concrete axial tensile strength
ε'_{bpl} compressive strain in concrete at peak stress f'_b or f'_{brep})
ε'_{bu} ultimate compressive strain in concrete

ε_{bu} ultimate tensile strain in concrete

E'_b modulus of elasticity for concrete

φ creep coefficient

Assuming that planes normal to the axis remain plain after bending, the bending moment can be determined for each axial force for a given concrete grade and wall section. For four different states of the cross section, the bending moment will be calculated from the equilibrium conditions for moment and axial force.

These four states are :

– cross section does not crack, nor yield=zone 1
– cross section cracks but does not yield=zone 2
– cross section cracks and yields=zone 3
– cross section does not crack but yields=zone 4

Each of these states corresponds with a zone in the interaction diagram, bordered by limit curves ABC, ODEB, BGHC and ODFH as shown in Figure 3.

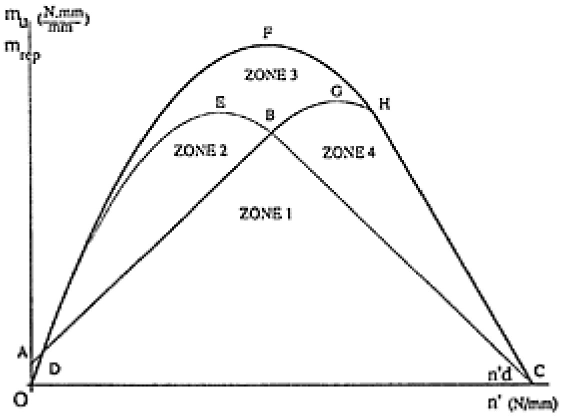

Figure 3. The axial force—bending moment interaction diagram

The curves are obtained from the conditions of equilibrium of a typical cross section. Equations have been worked out in the "Handbook for Plain Concrete SISMO® Walls" [2].

Computing A Facade Wall Of An N-storey Building

The plain concrete SISMO® walls are designed as braced structures. This means that they must satisfy the condition $m_d+m_{t.i.}\leq m_u$. It is sufficient to check this formula immediately above and below the floor slab because there md is maximal. If this condition cannot be satisfied, but $m_d\leq m_u$, it is still possible to find a position of equilibrium in most of the cases. This equilibrium exists if the rotation capacity of the wall (α_w) near to the wall-to-floor-connection is sufficient to follow the rotation of the floor slab (α_{pl}). Figure 4 shows the flowchart for assessing the cross section of a wall at the wall-to-floor connection.

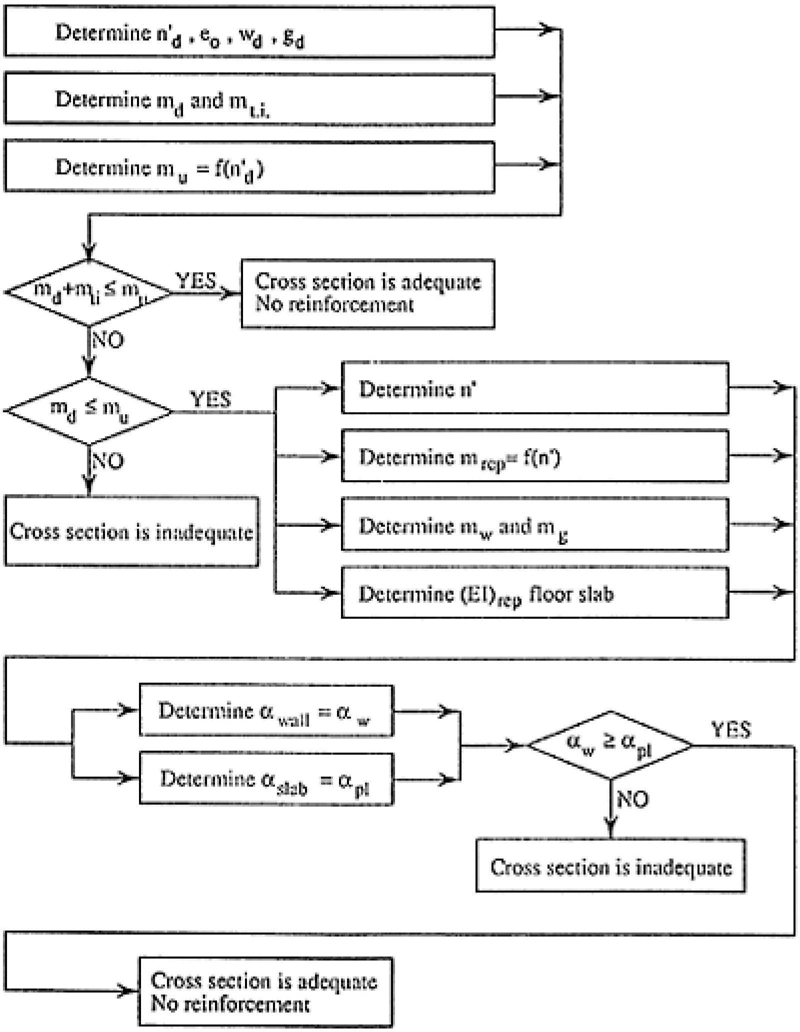

Figure 4. Flowchart for assessing the cross section of a wall at the wall-to-floor connection

where

e_o begin excentricity n'_d of
w_d design value of wind load

g_d design value of soil load

m_{wd} design value for the limit state of collapse, of the bending moment about the horizontal axis, due to wind load

m_{gd} design value for the limit state of collapse, of the bending moment about the horizontal axis, due to soil load

$(EI)_{rep}$ equivalent flexural stiffness of slab

As m_u and m_{rep} are given by the interaction diagrams, the only unknown so far is the rotation capacity of the wall α_w (α_{pl} is calculated using $(EI)_{rep}$ and the loads acting on the floor slab). To calculate the maximum α_w, the rotations of the wall segments immediately above and below the floor, resp. α^B and α^O are computed, applying at both ends of each wall segment the representative bending moments $m^{O}_{rep,i}$ and $m^{B}_{rep,i-1}$ given by the interaction diagram with regard to the limit state of serviceability, as shown in Figure 5.

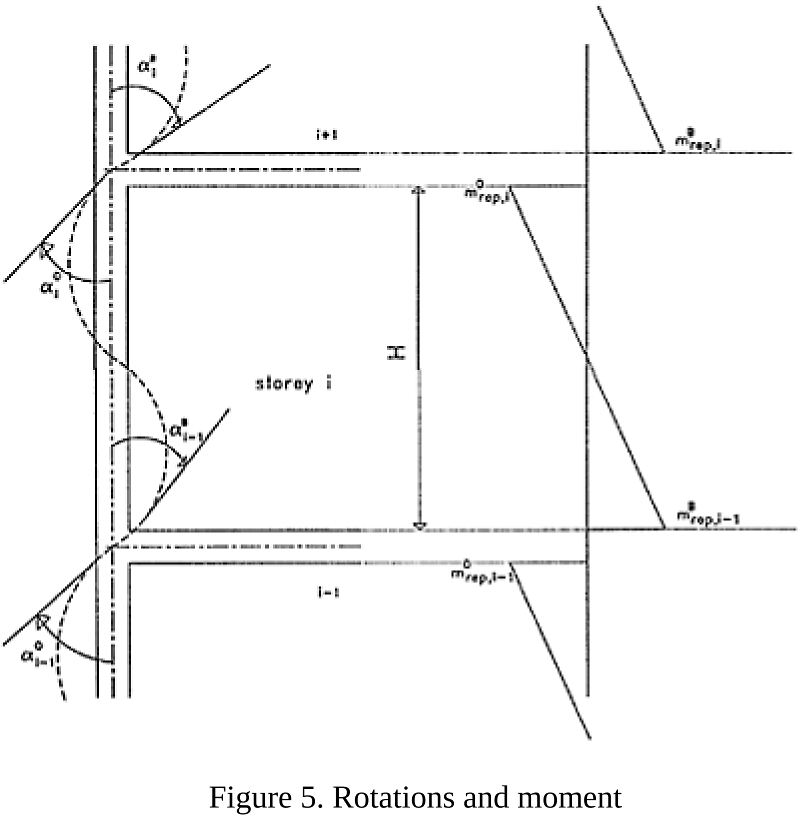

Figure 5. Rotations and moment distribution curves

The bending moment curve along the wall is either linear (no wind load) or parabolic (wind load or soil pressure). For the wall segment above and below the floor slab, the slope and deflection at any cross section are calculated, knowing the bending moment and axial force. By integration of the curvature $\kappa = (\varepsilon_b + \varepsilon_b')/y$, α^O and α^B are found, of which the smallest value represents the rotation capacity of the wall α_w.

To calculate α_{pl}, it is necessary to know which bending moment m_1 the floor slab maximally can apply at the wall for the limit state of serviceability. Assuming $\alpha^O < \alpha^B$, the value of the bending moment m^B can be found, so that the rotation of the floor-to-wall connection as a whole equals α^O. In that case $m_1 = m^O_{rep} + m^B$ The bending moment m^B can also be approximated in a safe way as $(\alpha^O / \alpha^B) \cdot m^B_{rep}$ (see Figure 6).

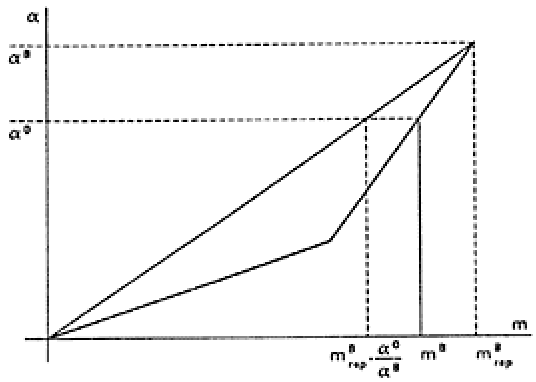

Figure 6. Safe approximation of m^B

Finally, if $\alpha_w \geq \alpha_{Pl}$, there is position of equilibrium for the wall-to-floor connection, so that the wall may be designed as a plain concrete element.

As shown in the “Handbook for Plain Concrete SISMO® Walls” [2], this calculation must be done for each possible load combination.

The study of an extensive number of examples proved that in most cases the conditions of equilibrium within the range of the appropriate stress—strain diagram, can be satisfied by a plain concrete section in both limit states (collapse and serviceability). Sometimes however the axial force n′ is too small to generate enough bending moment resisting capacity. This can only occur at the roof level : it is obvious that the maximum rotation capacity of a wall is obtained when the greatest possible number of cross sections crack and yield, i.e. when m_{rep} is defined by the ODFH-curve of the axial force-bending moment interaction diagram (Figure 3).

If the axial load n′ is smaller than n'_D, only the sections immediately above and below the floor slab will be able to crack and yield. All the other sections will deform elasticly. The cracks in the initially cracked sections will grow at those very sections until the structure collapses.

This is different from reinforced concrete where reinforcement spreads the crack over a zone in various small cracks. Therefore our (provisional and very safe) recommendation is to reinforce sections where $n' < n'_D$ by a traditional reinforcement limited to the wall-to-floor connection itself.

Since, however, experience proves that this reinforcement could be omitted, research is continued to explain this apparent anomaly.

EXPERIMENTAL STUDY

From the above we concluded that plain concrete can generally be used for walls and wall-to-floor connections.

One theoretical exception to this are the wall-to-floor connections where $n' < n'_D$, where we recommended reinforcement. In order to minimize the negative impact of this on the work on site, we developed easy to set SISMO® standard reinforcement for these connections, which can be calculated according to tested recommendations. The program of the experimental study is summarized in Tables 1 and 2.

The test program covered three subjects :

– The bending moment resisting capacity of a SISMO® wall-to-floor connection under vertical load (V)
– The bending moment resisting capacity of a SISMO® wall-to-floor connection under horizontal load (H)
– Preliminary tests on the shear strength of a SISMO® wall-to-wall connection (S)

Table 1. Tests on bending moment resisting capacity of a SISMO® wall-to-floor connection

Type	Nr	Reinforced / Plain walls	SISMO® type	Thickness wall (mm)	Thickness slab (mm)	Diameter reinforcement (mm)
V	1	R	250	150	150	10
V	2	PL	250	150	150	10
V	3	PL	250	150	150	14
H	1	R	250	150	150	10
H	2	PL	250	150	150	10

Concrete: C20/25 according to Eurocode 2 [3]
Steel: S500 according to Eurocode 2 [3]

Table 2. Tests on shear strength of a SISMO® wall-to-wall connection

Type	Nr	Thickness walls (mm)	Height walls (mm)	Ultimate shearing force (kN)	Diameter reinforcement (mm)*	Mean value concrete compr. strength f'_{bm} (N/mm^2)	Calculated shear strength ** (kN)
S	1	150	2.550	970	8	31,3	888
S	2	100	2.550	1.240	8	15	399
S	3	150	2.550	833	10	14	588

Reinforcement consists of horizontal hooked bars every 300 mm, steel quality S500 according to Eurocode 2 [3]
** Calculated shear strength $T = A_b \cdot \tau_b + A_s \cdot \tau_s$
Where
A_b=gross area of section
A_s=area of the reinforcement
τ_b=ultimate shear stress of concrete $\equiv 0{,}06 \cdot f'_{bm}$
τ_s=ultimate shear stress of steel $\equiv 0{,}85 \cdot f_{srep}$

A typical test set up can be seen in Figure 7.

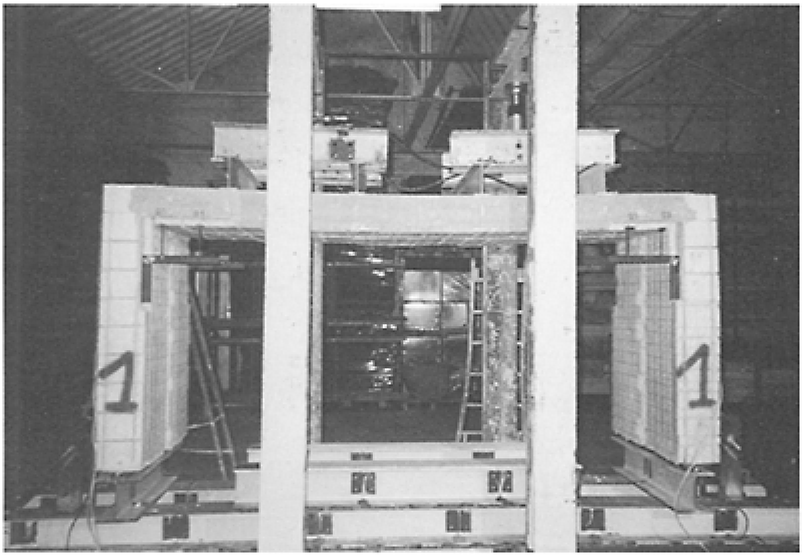

Figure 7.

It shows test piece V1 (wall-to-floor connection under vertical load) just after collapse of the right-hand wall-to-floor connection.

The SISMO® standard reinforcement of a wall-to-floor connection is shown at Figure 8.

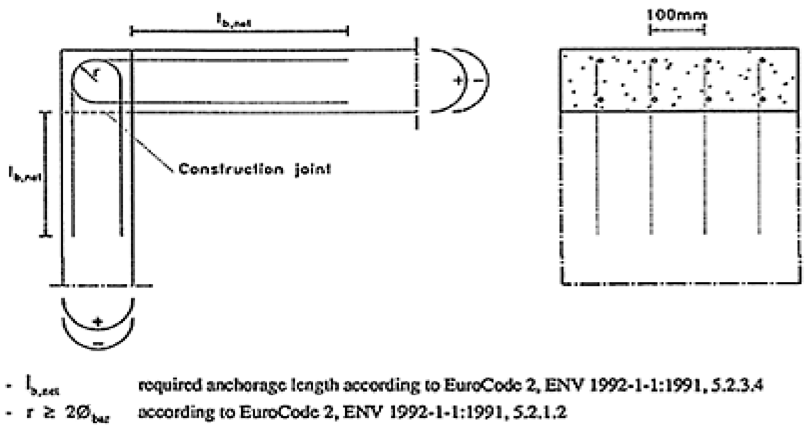

Figure 8. The SISMO® standard reinforcement of a wall-to-floor connection

The SISMO® standard reinforcement of a wall-to-wall connection is shown at Figure 9.

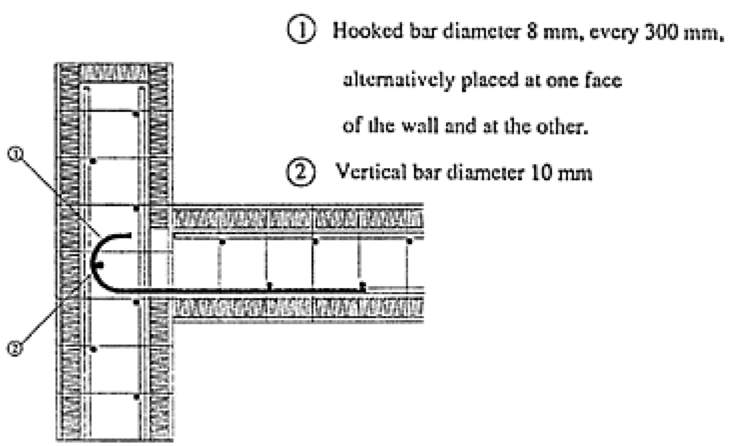

Figure 9. A SISMO® standard reinforcement diameter 8 mm, of a wall-to-wall connection

The results of the tests carried out on the bending moment resisting capacity of a SISMO® wall-to-floor connection are summarized at Table 3.

Table 3. Comparison of calculated and measured limit state bending moments

TEST		Cracking moment M_r * (kNm)		Moment of collapse M_c * (kNm)		
Nr.	Angle	Theoretical (1)	Measured (2)	Theoretical (3)	Measured (4)	M/T (5)=(4)/(3)
V1	Left	−21.652	−22.098	−61.048	–	–
	Right	−21.675	−23.418	−61.048	−55.407	0.91
V2	Left	−20.209	−21.379	−61.112	−61.606	1.01
	Right	−20.217	−19.695	−61.112	–	–
V3	Left	−23.460	−25.059	Structure collapsed through brittle failure of wall at end of angle reinforcement. The SISMO® standard reinforcement, diameter 14 mm is too strong for a plain concrete wall of 150 mm (see at Test nr. 3, 3.3)		
	Right	−23.451	−23.240			
H1	Left	−20.209	−22.737	−56.140	between −53.427 and −55.407	0.95 and 0.99
	Right	+20.206	+22.291	+51.094	greater than +56.040	>1.09
H2	Left	−20.202	−23.173	−56.204	between −54.308 and −61.606	0.97 and 1.10
	Right	+20.174	+24.038	+51.258	greater than +48.576	>0.95

Sign convention : +opening moment

−closing moment

Table 2 already gave the test results for the shear strength of SISMO® wall-to-wall connections as found in short span three point bending tests.

Test S1 showed a fair correspondence between experimental and theoretical (i.e. calculated) values.

Tests S2 and S3 however did not confirm that correspondence. Research is continued to find out the reason(s) for the different behaviour of those tests.

RECOMMENDATIONS FOR DESIGN OF WALL-TO-FLOOR CONNECTIONS WITH SISMO® STANDARD REINFORCEMENT

For a given percentage of reinforcement $\omega \equiv 1\%$, the bending moment resisting capacity, expressed as function of the theoretical moment of collapse of the weakest member of the wall-to-floor connection, is not less than 95% for a positive bending moment (closing moment) and 91% for a negative bending moment (opening moment). These conclusions confirm similar tests performed in Germany, that are reported in Beton-Kalender 1989, Part II, E, 4.4.

This leads to following recommendation :

The SISMO® standard reinforcement of wall-to-floor connections with $\omega \equiv 1\ \%$, (steel S500 and concrete C20/25 according to Eurocode 2 [3]), may be computed like a traditional, continuous angle reinforcement.

For this purpose an additional partial safety factor $\gamma_{s®}$ for loads, equal to 1,15, shall be applied on top of the normally used load safety factor(s).

This applies both for positive and negative bending moments.

where

ω, percentage of reinforcement $\equiv \frac{A_s}{A_b}$

$A_s \equiv$ total area of longitudinal reinforcement

$A_b \equiv$ gross area of section

N.B.

If the wall is designed as a plain concrete wall, the bending moment applied by the floor at the wall through the wall-to-floor connection shall be kept as small as possible. Therefore the percentage of reinforcement of the wall-to-floor connection shall not be greater than 1 %.

GENERAL CONCLUSIONS

The experimental and theoretical studies described in this paper show the great potential and flexibility of the SISMO® building system.

For braced structures design becomes extremely simple.

Computation of plain concrete walls can easily be done in a spreadsheet taking in account limit states of collapse and serviceability. When necessary, easy to place wallto-floor connection standard reinforcement can be designed according to experimentally confirmed recommendations.

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FACTORY AND SITE PRODUCTION—A COMPARISON

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Appropriate Concrete Technology. Edited by R K Dhir and M J McCarthy. Published in 1996 by E & FN Spon, 2–6 Boundary Row, London SE1 8HN, UK. ISBN 0419 21470 4.

ABSTRACT. The paper is a plea for the reconsideration of the benefits of prefabrication of concrete structures and suggests that the time is ripe for its increasing use.

Keywords: Precast Concrete, Prefabrication, Cladding, Quality, Speed, History.

Dr Howard P J Taylor is a Technical Director of Tarmac Precast Concrete Limited. He has many years of experience in this industry and constantly promotes the increasing use of prefabrication. He is a past President of the Institution of Structural Engineers.

INTRODUCTION

Prefabrication is one of our oldest and most successful construction techniques. In all forms of construction, apart from reinforced concrete and masonry, it dominates. On an industrial basis it is accepted in civil engineering, ship building, car and aircraft manufacture without question. For smaller equipment, including consumer goods and computers, modular methods of design and manufacture dominate. It is only in the building industry, perhaps with its tradition of hand craft work on site, with stone masons, bricklayers, carpenters, etc, that we find resistance to the good sense of its use.

Prefabrication has an intellectual justification in the thoughts and theories of the leaders of the industrial revolution. Adam smith expressed the benefits of the specialised work force and factory as:

> "This great increase of the quantity of work is owing to three different circumstances. Firstly, to the increase of dexterity in every particular workman; secondly, to the saving of the time which is commonly lost in passing from one species of work to another; and, lastly, to the invention of a great number of machines which facilitate and utilise labour and enable one man to do the work of many".

Adam Smith 1776
The Wealth of Nations

Adam Smith pointed out some of the advantages to be obtained from maximising factory based construction.

– Enhanced skill operatives
– Development of machines
– Speed

He left unsaid the other main advantage of precasting—

– Improvements in quality

HISTORICAL DEVELOPMENT

The Victorian Engineers took their lead from the earlier pioneers of the Industrial Revolution and tackled many projects in ways which were then innovative, to produce structures which could not be made without prefabrication.

Stephenson in 1845 moved the main box girders for the Britannia bridge across the Menai Straits on barges to be then jacked up on their final abutments. Brunel did the same thing with the tubular tied arch on the Tamar Bridge.

These massive structures were not manufactured in factories, but were assembled in safe locations—prefabricated—and moved to their final location. It was probably these spectacular successes that encouraged civil engineers to favour prefabrication. Concrete structures have also been similarly constructed and moved in a variety of ways. Transportation by floating—Mulberry Barbours and offshore oil platforms, by launching and by towing are all common.

DISCIPLINE OF PREFABRICATION

Prefabrication brings advantages of better "buildability" and speed in the whole structural range, from the most massive to the most simple. It is only necessary to visit a precast concrete frame structure being erected on site for one to appreciate the benefits of the dry form of construction, no deterioration of site stored materials and the absence of a large and often badly controlled workforce. The erection process from factory to truck to final location on site minimises opportunity for damage and for inferior work to be built in.

The description of Prefabrication also brings many benefits from an often disregarded opportunity in design.

– Advantages of the discipline

Some of the most inspiring works of human creativity comes from the skilled use of a limited language. In music for example a limited number of notes and instruments is capable of providing the most stirring of results. Indeed, there appears to be more beauty in the use of the musical language in a disciplined classical way than in more recent, freer use of sound. Prefabrication has this same opportunity, the observer notes the shape of the small components and appreciates how these can be built up with only minor modifications to a totally different, but readily understandable whole.

The aesthetic discipline can also pick up and emphasise, to advantage, a further practical necessity, the handling of joints.

DESIGN CONSIDERATIONS

In all forms of prefabrication the presence of joints brings three immediate requirements; the need to have sufficient strength, the need to avoid abrupt failure modes and the need to cope with manufacturing and construction tolerances.

– Strength of joints
– Durability
– Tolerance

From the aesthetic point of view a structure which is proud of its joints, in which joints are well articulated and detailed to express their function and load paths can be very successful.

In simple frame structures where competition with other materials is intense this may not be the case and demand is for a structure which does not have a precast "look". In bridges and other structures exposed to the external environment, joints must be detailed for maximum durability; sometimes they are expressed and sometimes not. When joints can be expressed, however, very attractive structures can result.

All structures have joints and these create local problems of concentrated forces and force resultants in the area immediately around the joint zone.

Saint Venant stated this principle clearly:

> "Forces applied at one point on an elastic structure will induce stresses which, except in a region close to that part, will depend almost certainly upon their resultant action and very little upon their distribution".

Away from joint zones, precast concrete design is no different from monolithic reinforced or prestressed concrete design. Even within joint zones the same rules for bearing and bond apply. A beam to column joint, column to column joint, or beam to beam joint, can all be understood in terms of internal struts and ties and a successful design will usually result if these concentrated forces and any further secondary forces from their interaction are satisfied. The only difference in precast construction is the additional problem of tolerances. It is easy to imagine how a physical displacement of a supported member from its correct support position can result in higher flexural forces on a cantilever bracket for example, for which it may not be designed. Similar displacements of reinforced members may result in there being no overlap of reinforcement for one member to the other. Joint design must always take account of the likely construction variations, whether intended or not.

These aspects of joint design are not unique to precast concrete; steel or timber construction also have similar problems which are, of course, solved in ways appropriate or each material. Bolted or welded steel connections always require a detailed analysis in the Saint Venant "zone".

ASPECTS OF CONSTRUCTION

The prediction of the development of machines has certainly come to pass in precast concrete factories. The development of machines to mix transport and place concrete has always been important. In the precast concrete industry the machines are becoming increasingly product specific, perhaps apart from highly engineered and architectural products reducing the precast manufacturer to the role of mere operator. In the concrete products area manufacturers all have similar machines and tend to compete in transportation and service as well as quality. It is this part of our industry where the importance of building a reputation related to brands is most clearly appreciated. Innovation in concrete products is now not just from the precast firm, but also from the machinery manufacturer.

The manufacture of engineered and architectural precast concrete certainly needs refined and well controlled equipment, but is also heavily reliant on engineering, management and craft skill. The development and retention of this skill was another of Adam Smith's predictions. The use of factory conditions and the retention of craftsmen now enables a quality of bricklaying and stone masons skill to be applied to structures in areas which would be prohibitively expensive, if not impossible to achieve on site.

The lead taken by Civil Engineers in welcoming prefabrication on site is now being taken up increasingly in the building industry. Structural components, frames, floors, stairs, cladding, etc have long been prefabricated. Attention is now being placed on the assembly of modular service pods, for risers, lifts and other heavily serviced areas. The prefabrication of service areas has been one of the great advances of the last decade and further work in this area promises more gains in speed and quality.

Our achievements in fast construction seem to be very great—but are they? A case study of one of the earliest large prefabricated structures brings out interesting parallels with prefabricated structures erected today.

In June 1850, Joseph Paxton, the Duke of Devonshire's Head Gardener, prepared a simple sketch proposal on blotting paper for the hall to be used to house the Great Exhibition in Hyde Park, due to open in May 1851. This was an alternative design, but was, nevertheless, accepted by the Exhibition Committee, which had already rejected the original official design. This started the race to design, detail, fabricate, erect and fit out one of the World's greatest prefabricated structures. Paxton was by no means unused to construction of large projects. He had already built some massive greenhouses for the Duke of Devonshire at Chatsworth, one of which, the Great Conservatory, at 82 meters in length, 37 in width and 20 metres high, was the largest glasshouse in the world. From that to the Crystal Palace with its dimensions of 564 metres×124 metres×33 metres was simply a practical difficulty!

The contractor, Cox Henderson, was taken on board in early July 1850 and the first column was erected in mid August. A period of only 10 weeks from concept design to start on site is more than impressive, even to this day. The programme for detailed design and construction of nine months was achieved and in that time 92,000 sq m^2 of building, or 1.2 million m^3 of volume was enclosed with a structure comprising cast iron columns, cast and wrought iron girders, timber girders and floors and glass. More than 2,000 men were employed in Hyde Park at the peak of building and fit out.

Typical erection speeds for precast frames are nowadays $1000m^2$ per week per crane with a site team of 8 men. Three teams with three cranes could, therefore, erect $92,000m^2$ in 30 weeks (fig.1). The time is the same, then as now, but now with the labour requirements dramatically reduced. As another measure, a highbay warehouse using double tees as wall and roof units to dimensions 156m×16m× 16m was erected in 1982 in 17 working days with a single crane. Thus, $2400m^3$ was enclosed per day (fig.2).

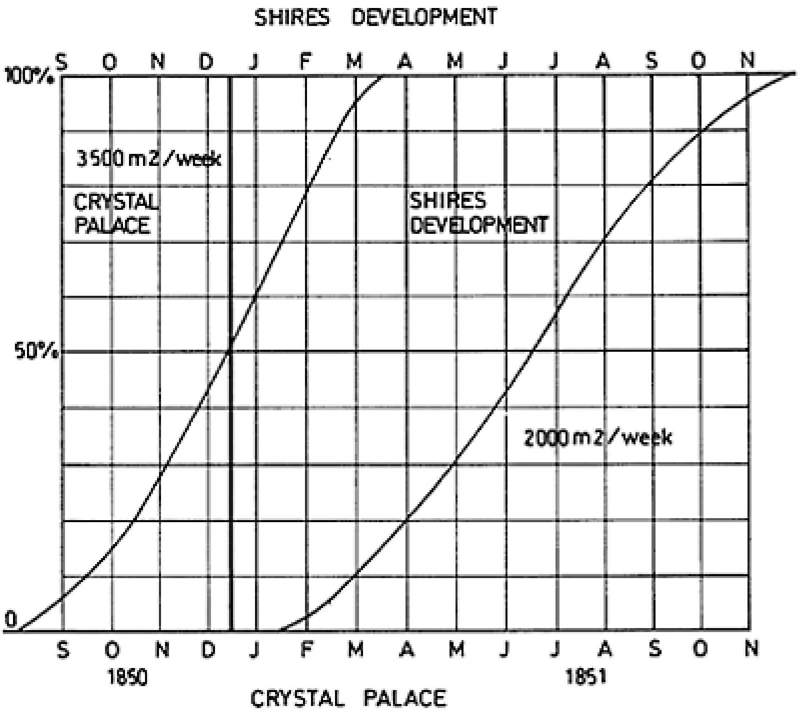

Figure 1 Speed of erection. Concrete frame v Crystal Palace

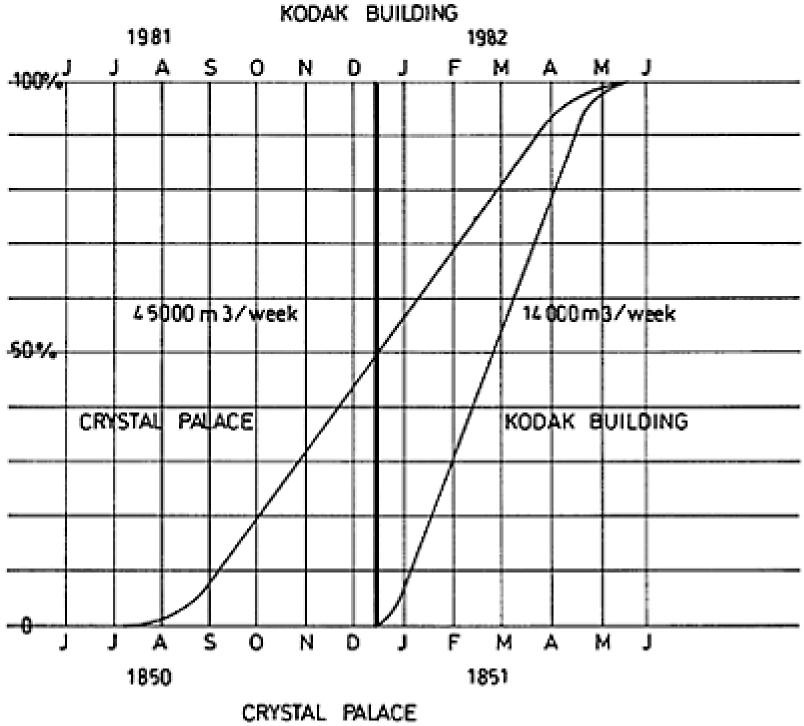

Figure 2 Speed of erection. Kodak Building v Crystal Palace

The Crystal Palace structure enclosed some 1.2 million m^3 in 210 working days at a rate of $5700m^3$ per day. The building rate at the Crystal Palace was clearly very fast, but it would be easily achievable to this day with three or four experienced erection teams of 6–10 men each.

The achievement of speed in design and procurement was remarkable and it is difficult to see how it could be matched now even for such a regular design to an established system in a green field site.

Some of the details of the design were equally remarkable and show interesting parallels with modern structures. The glazing support beams of the Crystal Palace were composite, iron and timber. The timber compression block was profiled to collect water from the glass and channel it to gutter beams which in turn passed rainwater to the columns, which passed it on to drains through holes in their centres. Small grooves were also cut in the side of the gutter block beneath the glazing to take condensation from the underside of the glass down to the main gutters. In erection, the gutter beams were used to support a wheeled gantry from which men placed and sealed the glass.

A similar detail has been used in cantilever roof beams in a number of grandstands in the UK and elsewhere, designed by the Jan Bobrowski Partnership. In this case too the gutter was used to support a wheeled erection gantry for the roof sheeting.

Another similarity was more tragic, the floor of the Crystal Palace was boarded with spaced timber planks so that every night it could easily be cleaned by brushing all the litter through to the void beneath. It was the accumulation of this material that contributed to the severity of the fire that eventually destroyed the building in 1936, which by then had been re-located in South London.

Recent fires at Bradford Football Club and at Kings Cross Underground Station, which both had a tragic loss of life, were similarly fed with the accumulation of debris beneath the structures.

Adam Smith’s final predication was on quality. This has already been mentioned, but deserves re-emphasis; for both speed and quality of finish, precast cladding is hard to match. Modern brick facades are becoming more ornamental and using prefabricated techniques, brick panels with corbelling and bricks applied to horizontal soffits are all possible. Combinations of natural and reconstructed stone in panellised form also now have produced a new revival of attractive traditional elevations in buildings.

Far from being an offshoot of the regularity of dull architectural modernism the prefabrication of concrete structures is developing into well engineered systems involving external envelope, often using natural materials, structure and services. Off site quality is now being integrated speedily into an attractive consistent whole.

CONCLUDING REMARKS

BARRIERS TO GREATER USE OF PREFABRICATION

With the positive message given in this paper, why is precast concrete not used much more widely? The answer to this question lies in a consideration of culture and cost.

The culture of our UK industry is for the design professions to have the main contact with the client and to feed client wishes to contractors in the form of designs and specifications. There is a clear self interest for this approach to continue. It is constructive to consider methods of procurement in other countries where Clients talk directly to Contractors and where the Designers are often in bureaux owned by the Contractors. The debate is not straight forward, the design professions rightly claim that their independence militates towards good design and good advice, although there is now an increasing realisation that with the few structures now being built and fee competition in design, this quality of service may not be universally provided.

On the cost side, the issue is more straightforward. For well integrated prefabrication techniques and systems, bridge beams for example, in all economic climates prefabrication is an economical option.

In the building industry the solution is slightly different. Despite the better production from mechanised factories, the mature, skilled labour force is normally paid at a sensible stable rate. On site, in times of boom, as was experienced in the late 1980’s, labour rates can literally go through the roof. This drove a great number of Contractors to favour prefabrication often carried out by re-designing work already designed in steel or insitu concrete. In hard times factory labour cannot easily be laid off and will earn similar rates to those in the boom. Site labour costs, however, plummet and suddenly all sorts of work, that for other reasons may be required to be precast, is converted to insitu.

The conclusion to all of the considerations in the paper is that the precast industry would be best advised to promote its benefits directly to clients. The new awareness of major clients of the benefits of engaging in dialogue and in partnering with suppliers is a positive move in this direction.

DEVELOPMENT OF SELF-CURE CONCRETE

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ABSTRACT. Since curing of concrete is seldom carried out to an adequate extent, due to its time consuming nature, there is a distinct need for the development of an admixture-based internal curing system. Work has been carried out to identify a chemical admixture that will improve water retention in concrete, leading to a higher degree of hydration and improved properties. Such a chemical has been identified and its addition to concrete leads to reduced drying shrinkage, and good surface quality. This improvement in surface quality leads to a high resistance to the ingress of chloride ions, and good carbonation characteristics, making it far better at protecting reinforcement from corrosion, compared to air cured controls. Whilst no improvement in concrete bulk strength is possible, the improvements in surface quality and general permeability are advantageous.

Keywords: Curing, Self-Cure, Admixture, Shrinkage, Permeability, Chloride, Carbonation, Corrosion.

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Thomas D Dyer is a research student at the University of Dundee. He is currently completing his PhD on the subject of Self-Cure Concrete. He specialises in the fields of cement chemistry, microstructure and concrete durability.

INTRODUCTION

Quality of cure is widely recognized as being a major factor in determining a concrete structure's subsequent performance. There are a number of curing techniques which either reduce the loss of moisture, or provide an additional supply at early ages. However, these methods can introduce problems to the construction process. Curing membranes can leave the concrete surface with poor adhesion properties, and it is often necessary to remove them subsequently using organic solvents. Plastic sheeting can be blown away by strong wind, leaving the concrete vulnerable to drying. However, the major problem with all curing techniques is that they are time consuming, and are, consequently, seldom carried out to an adequate extent.

There is, therefore, a genuine need for a means of curing concrete which requires minimum effort after placing. An ideal solution would be a 'self-curing' admixture that when added to the mix would act to improve the retention of water in the concrete from the inside.

IDENTIFICATION OF SUITABLE CHEMICAL ADMIXTURES

It was decided that the best way to enhance water retention in concrete using a chemical addition was by exploiting a deviation from Raoult's law displayed by certain non-ideal solutions. Raoult's law states that the chance of finding a molecule of a component in the vapour above an ideal solution is equal to the mole fraction of the component in the solution itself [1], and is described by the equation below:

$$p_A = x_A p_A^*$$

where P_A is the partial vapour pressure of a component A in an ideal solution, the mole fraction of A is represented by x_A, and P^*_A is the vapour pressure of pure A. This relationship is shown in Figure 1(a). This is only true of a closed system, and in the case of concrete, P_A will give an indication of the water's tendency to evaporate into the atmosphere.

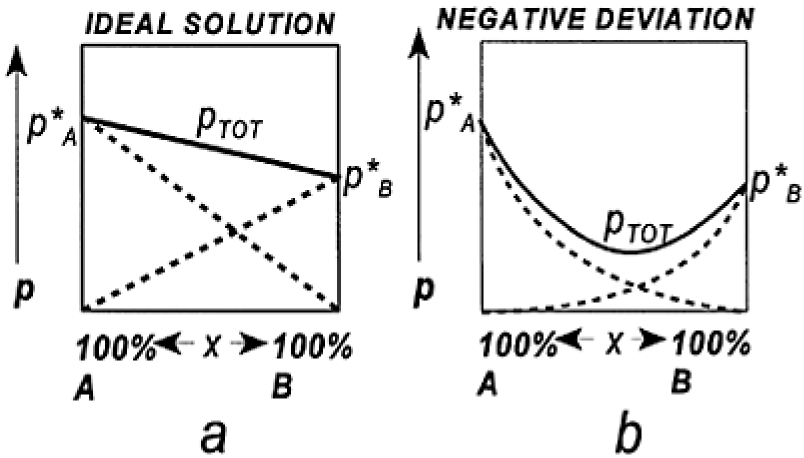

Figure 1. Vapour pressures above (a) an ideal solution and (b) a solution with attractive forces between molecules of the two components.

There are no attractive or repulsive forces between molecules of different components in an ideal solution. However, in real solutions this is not the case. When attractive forces exist between molecules there is said to be a 'negative deviation' from Raoult's law. This situation is illustrated in Figure 1(b) where the partial vapour pressures of the components and the total vapour pressure of the solution are reduced below those of an ideal solution. It is this deviation from Raoult's law that was hoped to be exploitable in developing self-cure concrete.

Therefore, the identification of chemicals capable of producing a negative deviation in water was required. One attractive force that water molecules can exert is that produced by hydrogen bonding. This interaction is caused when water molecules, whose hydrogen atoms possess a small positive charge, become electrostatically attracted to electronegative atoms. The attraction is particularly strong with electronegative atoms possessing lone-pairs of electrons, such as oxygen. Therefore hydroxyl (-OH) and ether (-O-) functional groups are particularly effective at taking part in hydrogen bonding with water (Figure 2). For this reason, chemicals possessing these functional groups were selected as potential self-cure admixtures [2]. Some of these chemicals are described in Table 1.

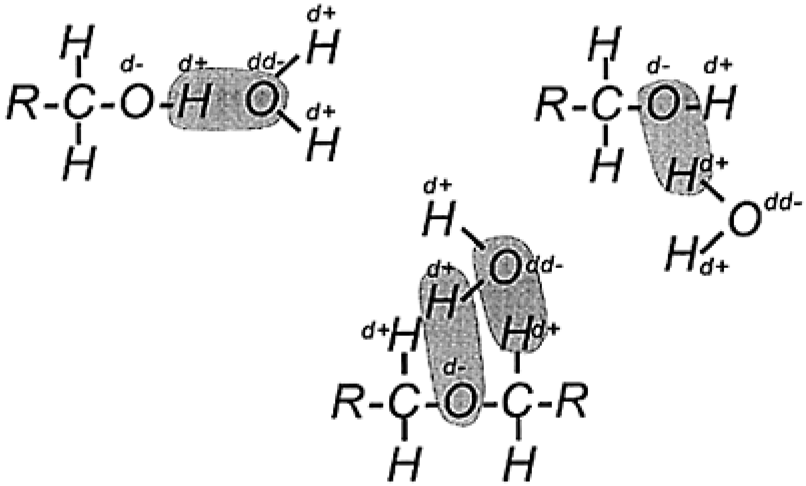

Figure 2. Examples of hydrogen bonds between functional groups and water molecules.

Three criteria were identified for the effects of a chemical addition to a cement paste to be described as ‘self-curing’ :

- When cement pastes containing the chemical are cured in air they should display improved water retention with respect to identically cured controls.
- As a result of any improvement in water retention, the degree of cement hydration should also be enhanced.
- An increase in the paste’s compressive strength with respect to air cured controls should be observed.

Table 1. Main characteristics of polymeric materials

	Main Characteristics of Polymeric Materials			
Chemical	**Description**	**Maximum Solubility at 20°C**	**Functional Groups**	
			-OH Groups	**-OGroups**
A1	**Synthetic. Av.M.wt. 200**	**100**	**Yes**	**Yes**
A2	**Synthetic. Av.M.wt. 1500**	**100**	**Yes**	**Yes**
A3	**Synthetic. Av.M.wt. 5000**	**100**	**Yes**	**Yes**
A4	**Synthetic. Av.M.wt.**	**60**	**No**	**Yes**

	10000			
A5	**Synthetic. Av.M.wt. 20000**	**65**	**Yes**	**Yes**
A6	**Natural. Av.M.wt. >20000**	**5**	**Yes**	**Yes**

To establish which of the selected chemicals complied with these criteria, a series of cement paste tests were carried out. The findings of these tests are shown in Figure 3, which shows the improvements in water retention (determined by measuring weight-loss), degree of hydration of the alite phase (measured using XRD) and compressive strength observed in 28 day old Portland cement pastes.

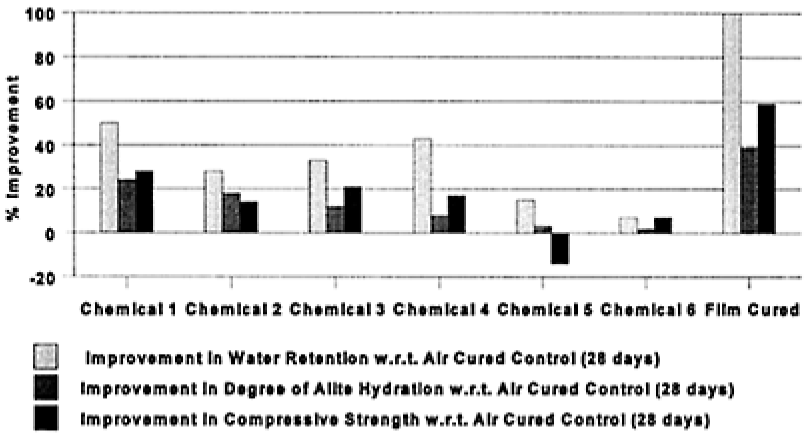

Figure 3. Improvements in cement paste properties produced by additions of some of the selected chemicals (with respect to air cured controls)

It is clear from the results that an improvement in cement paste water retention does not necessarily produce a proportional increase in the degree of hydration. Indeed it would appear that the degree of alite hydration decreases with an increase in average molecular weight. Improvements in compressive strength seem also to be, in many cases, influenced by more than just increased water retention. Chemical A1 performs the role of a self-cure admixture to the best extent and it is the performance of this chemical that will be discussed subsequently. It has been established that there is, in fact, a group of polymers similar to chemical A1 that all have similar properties.

PROPERTIES OF SELF-CURE CONCRETE

Hydrated Cement Microstructure in Self-Cure Concrete

Figure 4(a) is a micrograph from the SEM showing calcium hydroxide crystals at the interface between the cement paste matrix and an aggregate particle (the aggregate surface can be seen showing through beneath) in an ordinary mortar made using Portland cement as the binder. The effect of dosing the mortar with self-cure admixture can be seen in a similar micrograph in Figure 4(b). The calcium hydroxide crystals are considerably thinner and, it would appear, more numerous. This is probably a result of molecules of the admixture retarding growth of these crystals.

There is a distinct possibility that, as well as the reduction of vapour pressure, this microstructural, change is one of the factors that gives self-cure concrete such efficient water retention properties, since it would most likely hinder the escape of water vapour to the surface.

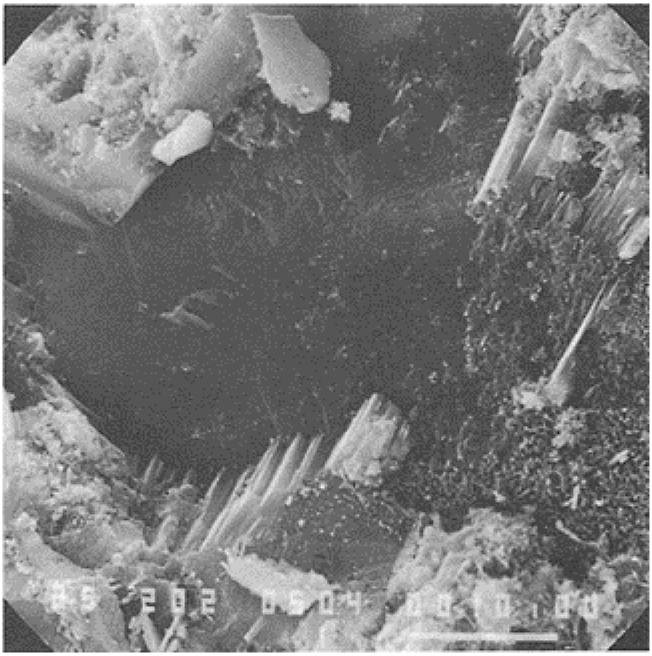

Figure 4(a) Calcium hydroxide on the surface of aggregate particles in an ordinary mortar

Figure 4(b) Calcium hydroxide in a mortar containing 0.1 00M self-cure admixture.

Shrinkage

Drying of concrete leads to shrinkage. Whilst concrete will always shrink to a certain extent after placing as a result of the uptake of water during hydraulic cement reactions, evaporation has the effect of greatly increasing shrinkage. This can lead to dimensional problems and, especially at early ages, cracking. Figure 5 shows the dimensional changes displayed by three Portland cement paste prisms up to an age of 180 days whilst kept in a 20°C/60%RH environment. Two controls were used: one sealed in film for the duration of the test and one exposed to the air. The self-cure specimen was made from paste containing a 0.100M dosage of admixture.

For up to around the first 28 days the self-cure specimen displays shrinkage behaviour similar to that of the film cured control. After this age its shrinkage increases at a greater rate than the sealed specimen. This is presumably because, in the film cured specimen, by 28 days the majority of hydration reactions have ceased. Therefore, beyond this point shrinkage is almost solely due to water evaporation, which will not occur in the sealed prism. At all times it is clear that the performance of the self-cure specimen is better than the air cured control.

Surface Quality

The results of 10 minute initial surface absorption tests (ISAT) [3] conducted on self-cure concrete at 28 days are shown in Figure 6. Concrete mixes containing 100% PC, 40% granulated blast furnace slag (GBS)/60% PC and 30% pulverised fuel ash (PFA)/70% PC were used. Results for both film and air cured controls are also shown. Film curing provides the highest quality surface, although the higher dosage of self-cure admixture gives very good results. Both self-cure mixes display better surface characteristics than the air cured control.

It is possible that the microstructural changes produced by the self-cure admixture also affect surface permeability. However, the majority of the improved surface quality is most likely due to the improvement in cement hydration compared to the air cured control.

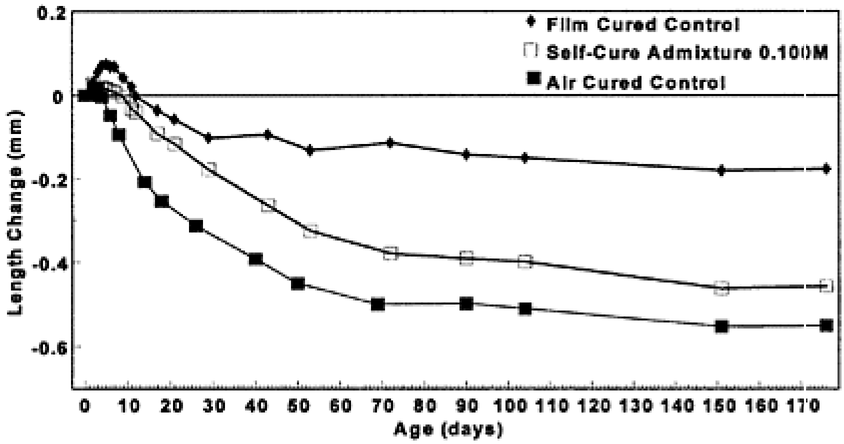

Figure 5. Shrinkage of Portland cement paste prisms kept in a 20°C curing environment

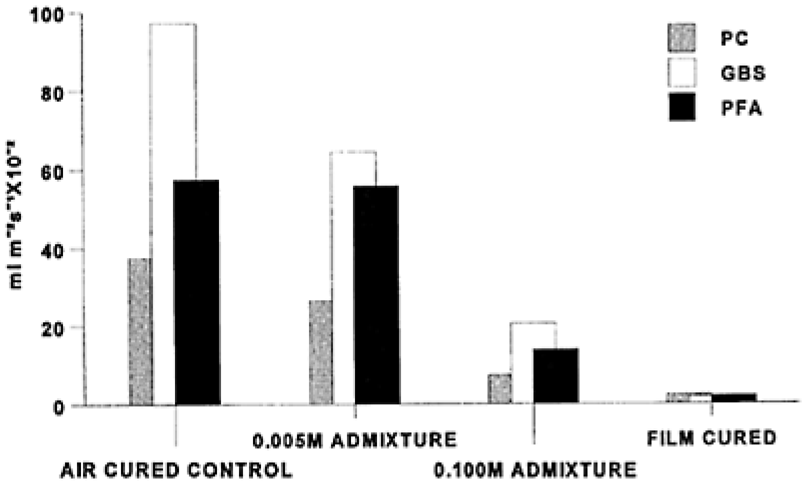

Figure 6. 10 minute ISAT readings for 150mm concrete cubes at 28 days.

Chloride Ion Penetration, Carbonation and Corrosion.

A curing-dependent aspect of concrete durability of great concern is the corrosion of steel reinforcement. Corrosion is generally caused by the ingress of harmful chemical species, such as chloride ions, from the concrete surface. It is, therefore, important that the concrete cover provides a good resistance to penetration by such substances. The corrosive capabilities of these chemical species is somewhat lessened by the pacifying effect of the high pH cement matrix around the steel. However, the pH can be reduced by reaction with atmospheric carbon dioxide, and so it must also be ensured that the concrete provides protection against carbonation. Since the concrete surface is the region most sensitive to curing, resistance to chloride ion ingress and carbonation are both heavily influenced by the quality of cure.

Figure 7 shows the results of chemical analyses conducted on 5mm slices taken from 70mm cement paste cubes which have had one face exposed to a cyclical sodium chloride solution spraying regime for 3 months. The pastes all had w/c ratios of 0.5 and were 100% PC, 40% GGBS/60% PC, or 30% PFA/70% PC. Curing was conducted for 28 days prior to exposure in a 20°C/60%RH atmosphere. A 0.1 00M dosage of self-cure admixture was used in one of each type of paste mix, whilst one of the undosed pastes was sealed in film during the curing period. The exposure regime consisted of 12 hours of spraying, followed by a 12 hour pause.

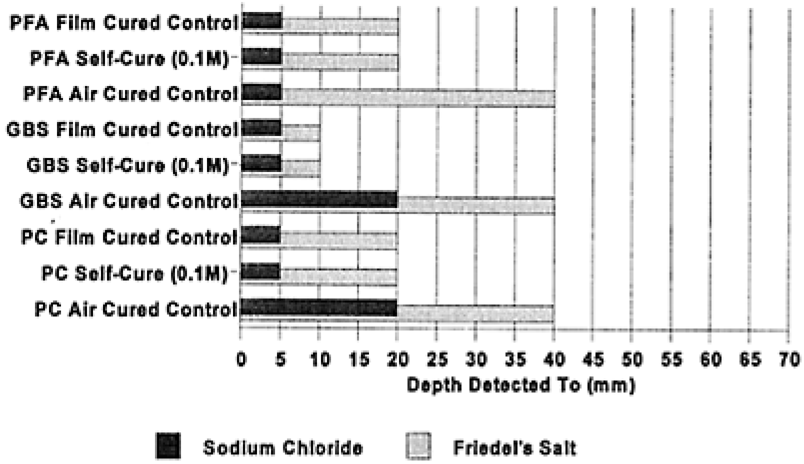

Figure 7. Friedel’s salt and sodium chloride profiles of 70mm cement cubes exposed to 3 months of cyclical chloride solution spraying.

The slices were dried in air, ground in a ball mill and analysed using both TG/DTA and XRD techniques. Figure 7 shows the depths to which sodium chloride and Friedel’s salt were detected. Friedel’s salt is the low chloride content calcium chloro-aluminate ($C_3A.CaCl_2.10H_2O$) and results from the reaction of chloride ions with the C_3A and C_4AF cement phases and their hydration products [4]. This reaction renders the chloride ions immobile at pH levels experienced in most uncarbonated, cement matrices [5]. It is clear that the depth of penetration of chlorides into the pastes is much lower when film curing is used, compared to the air cured control. However, results virtually identical to the film cured control can also be seen in the self-cure pastes.

The depth to which carbonation has occurred such that the cement matrix is rendered ineffective in pacifying corrosion can be assessed by spraying a solution of phenolphthalein onto a freshly fractured concrete surface [6]. The results of carrying out this procedure on concrete specimens kept in a carbonating environment for 2 weeks is shown in Figure 8. Again it is apparent that the addition of the self-cure admixture has a similar effect on carbonation behaviour as film curing.

Figure 9 shows the corrosion potential of steel reinforcement in cubes kept in the same chloride spraying environment described previously. Corrosion potential was measured using the half-cell potential technique [7]. As would be expected from the previous findings, the film cured control displays the lowest potential to corrode, whilst the air cured control shows the greatest. As self-cure admixture dosage increases, the potential for reinforcement to corrode is lessened, due to the improved surface qualities and overall reduction in permeability. It should be noted that in this case the concrete is not likely to have carbonated to any great extent and that the observed corrosion potential is, therefore, almost exclusively due to chloride ingress alone.

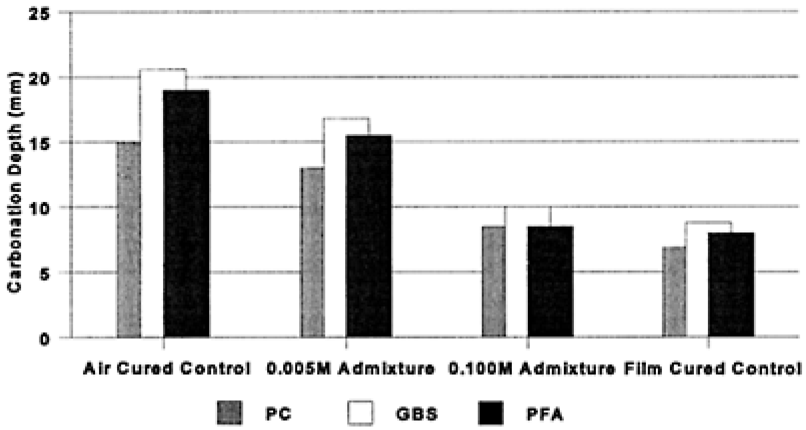

Figure 8. Depth of carbonation after 2 weeks exposure to a carbonating environment.

Strength

Whilst, in general, the properties of self-cure concrete show an improvement in comparison to air cured controls, an increase in compressive strength is not observed. Since cement paste strength is increased by the addition of the admixture it is clear that it is the presence of aggregate that is responsible for the lack of improvement in concrete strength. The most likely reason for this is the change in calcium hydroxide morphology discussed previously. It is reasonable to assume that the thinner calcium hydroxide crystals will be weaker, and since it is these crystals which contributes to the bonding between the cement paste and aggregate particles, the strength of the bond will be lessened.

Whilst this is situation is obviously less than ideal it should be noted that the improvement in surface quality far out measures the lack of improvement in strength: the permeability per unit strength for self-cure concrete is always better than that of air cured controls [8].

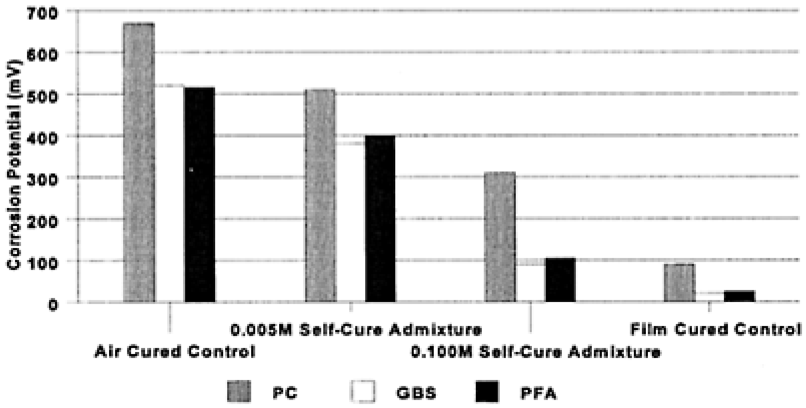

Figure 9. Half-cell corrosion potential after 28 days chloride spray exposure

CONCLUSIONS

1. The addition of a suitable chemical admixture can improve water retention in hardening concrete and, as a consequence, increase the degree of hydration in cement pastes cured in air compared to identically cured controls.
2. The manner in which self-cure pastes shrink is close to film cured controls up until around 28 days. However, beyond this age the extent of shrinkage gradually approaches that of the air cured control.
3. Improvements in the degree of hydration in air cured concrete have the effect of improving surface quality. Consequently, both resistance to chloride ingress and carbonation are improved in comparison to air cured controls, making self-cure concrete effective in protection of steel reinforcement from corrosion.
4. Whilst an improvement in concrete strength with respect to air cured controls cannot be expected in self-cure concrete, its other qualities far outweigh this disadvantage.

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VIBRATORS FOR COMPACTING CONCRETE—A RADICAL LOOK

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ABSTRACT. Application of vibration, of appropriate amplitude and frequency, to fresh concrete reduces its yield value to zero and permits it to flow under its own weight to pass between reinforcement, fill formwork and release air bubbles. Previous work suggested that the peak velocity is the most important characteristic of the vibration and a re-examination of results obtained in a vertical pipe apparatus confirms that there is a linear relationship between fluidity of the vibrated concrete and the peak velocity (given by amplitude x frequency) of the vibration. The proportionality constant is termed the vibrational susceptibility and is characteristic of the material, but decreases as frequency increases. The greatest fluidity and hence the most rapid placement of concrete is therefore achieved at low frequencies (16–30 Hz) rather than at the frequencies commonly employed in industrial vibrators for concrete (50–200 Hz).

Keywords: Fresh concrete, rheology, vibration

Professor Phillip F.G.Banfill is Professor of Construction Materials, HeriotWatt University, Edinburgh, UK. A chemist, his research experience is in the chemical and related properties of cement based materials, including rheology and durability, and he is the author and co-author of 3 books and 60 papers on various aspects of the behaviour of construction materials.

INTRODUCTION

Vibration has long been recognised as necessary for effective compaction of concrete and arising from much early work done on the effect of vibration on the properties of the hardened concrete recommendations for current practice exist in many countries. The general consensus seems to be that the higher the acceleration at moderate frequencies the more effective the vibration. However, it is much more fundamentally sound to base recommendations on the performance of fresh concrete and advances in the understanding of the rheology of fresh concrete have permitted this. This paper discusses this approach and some important implications.

EFFECT OF VIBRATION ON HARDENED PROPERTIES

The importance of vibration as a means of compacting concrete has been recognised for a very long time and much progress was made as a result of early studies of the effect of vibration on hardened properties [1, 2, 3], which are embodied in current practice recommendations e.g. [4]. The application of vibration, of an appropriate frequency and amplitude, fluidifies the concrete enabling air pockets to be filled and air to bubble to the surface. An initial gross increase in bulk density is observed, after which prolonged vibration permits slow reorientation and packing of the particles in the concrete. The main consequence of the increased density is higher strength. The air content of the freshly mixed concrete may exceed 10% by volume and removal of this amount of air from a test cube by vibration will double the measured compressive strength. Associated with the strength increase is the achievement of the other desirable hardened properties.

RHEOLOGY OF FRESH CONCRETE

With advances in the understanding of its rheology came the possibility of studying the effect of vibration upon fresh concrete. It is now well established that fresh concrete, over the range of shear rates important in practice, conforms to the Bingham model and that the yield value and plastic viscosity can be measured conveniently in the two-point workability apparatus [5]. In the test the torque required to turn an impeller at several speeds in a bowl of fresh concrete is measured. From the resulting data a graph of torque T against speed N shows a straight line relationship

$$T=g+hN$$

where g and h can be shown theoretically to be proportional to yield value and plastic viscosity, respectively, the Bingham constants of the material. In practical terms, the fact that fresh concrete has a yield value explains why it can support its own weight, as for example in the slump test. Under vibration concrete loses that ability to support itself and flows freely under its own weight to pass between reinforcing bars and fill the corners of a mould. This suggests that the vibration either applies sufficient shear stress to exceed the yield value or causes a change in the nature of the material such that the yield value is reduced to a low level. Extensive work summarised by Tattersall [6] confirmed that the latter was the case.

EFFECT OF VIBRATION ON RHEOLOGY OF FRESH CONCRETE

Tattersall and Baker [7] mounted the bowl of the two-point workability apparatus on an electromagnetic vibrator and constructed flow curves with and without vibration over a range of frequencies and amplitudes. They found that the flow curve was instantaneously and reversibly altered by the application of vibration and that the yield value decreased to zero (Figure 1), i.e. the constructed flow curve fits a power law model passing through the origin. This enables the concrete to flow under its own weight. Consideration of the region near the origin in Figure 1 suggests that, under vibration, and at low shear rates, fresh concrete behaves as a Newtonian liquid. This means that the viscosity of the vibrated concrete or its inverse, the fluidity, will be the key parameter in determining how it will flow.

INFLUENCE OF VIBRATIONAL PARAMETERS

Assessing the influence of vibrational parameters on the rheology is impossibly complex when the unvibrated and vibrated curves conform to different models (Bingham and power law), so Tattersall and Baker [7] found it necessary to approximate the unvibrated curve to a power law of the form

$$T=\alpha\ N^{\beta}$$

It was then possible to study the effects of vibration by considering changes in the ratio α_v/α and the difference $(\beta_v-\beta)$, where the subscript denotes the value determined under vibration. They found that, subject to exceeding a small threshold frequency, the most important characteristic of the vibration to be its peak velocity.

Constructing a flow curve from individual readings of torque and speed is cumbersome and the data analysis is complex, so Tattersall and Baker concentrated on the region near the origin of the flow curve where the slope of the curve gives a measure of the fluidity. This suggested that a simpler apparatus could be successful and in a comprehensive series of tests using a much more convenient vertical pipe apparatus they determined the fluidity of the concrete from the rate of efflux from the bottom of the pipe into the vibrating receptacle [8]. Under vibration, the height H of the column of concrete in the pipe decreases with time as concrete flows from the bottom according to

$$dH_{/dt}=-bH$$

where b is a constant proportional to the fluidity.

Based upon measurements on four concretes carried out at 11 frequencies between 16 and 200 Hz and 8 accelerations between 0.85g and 8.90g, the effect of vibrational parameters—frequency f and amplitude A—on b can be modelled by the equation

$$b=1_{/c}\ \ln\ (1-f_{/F}).[A-A_o]$$

where F is an upper limit of frequency (between 250 and 500 Hz), A_o is a threshold amplitude, which in turn depends on frequency, and c is a constant [8]. This equation shows that there is a threshold amplitude below which vibration has no effect in that yield value is not reduced sufficiently for flow to occur, and that the effect of vibration decreases markedly as frequency approaches some upper value F. If $A>>A_o$ and $f<<F$ this equation reduces to

$$b=KAf=Kv$$

since peak velocity v is the product of amplitude and frequency. The constant K is characteristic of a particular mix and may be termed the vibrational susceptibility. It may

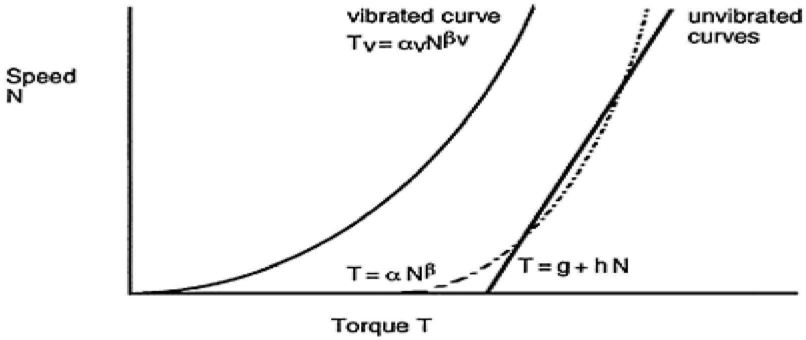

Figure 1 Row curves for unvibrated and vibrated fresh concrete.

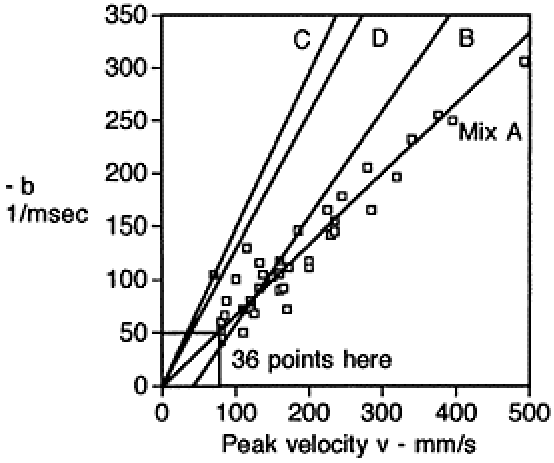

Figure 2 Replot of data showing the relationship between -b and v [8]. Points are shown for mix A only. All mixes 20–25 mm slump.

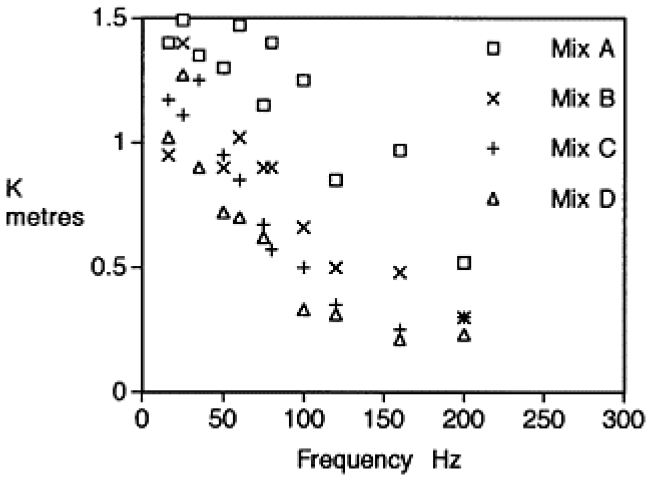

Figure 3 Relationship between vibrational susceptibility K and frequency.

be understood as follows. The larger the value of K, the larger the value of b and the faster the concrete flows out of the pipe when it is subjected to vibration of peak velocity v, as would be expected of concrete with a higher vibrational susceptibility.

Effect of peak velocity on vibrational susceptibility

The validity of this relationship can be tested by replotting Tattersall and Baker's original data for mix A in the form of -b against v as shown by Figure 2. It is reasonable to expect there to be a significant effect of material parameters, in particular the general level of workability, on the behaviour of concrete under vibration. Accordingly, also plotted in figure 2 are the lines of best fit for their mixes B, C and D, but all their mixes were prepared at similar workabilities (20–25mm slump) and the scatter of points means that no significant trends are discernible. However, differences are shown by the results of Wood [9] and Millar [10], who worked with the same apparatus but with more workable concrete of 100mm slump. Instead of the values of K in the range 0.75–1.5 shown by the lines in Figure 2 they obtained, for four different concretes at this higher workability, K in the range 1.6–2.1. This shows that the fluidity under vibration was higher, i.e. K had changed in the sense expected. This suggests that the vibrational susceptibility is a material parameter and is influenced by the general level of workability of the concrete, although much more work will be needed before the nature of that relationship can be established. Some initial work to explore the effect of rheology on K is in progress.

Effect of frequency on vibrational susceptibility

For vibrational susceptibility to be a true material parameter its value should be independent of the vibration conditions. Since Tattersall and Baker tested their concretes at up to seven levels of acceleration for each frequency, graphs of -b against v plotted from their original data at each frequency have up to seven points—sufficient to define the value of K. Figure 3 shows the resulting effect of frequency on vibrational susceptibility. Despite the method of data treatment causing the values of K to cover a wide range for the four mixes the broad trend is quite obvious: vibrational susceptibility decreases with increasing frequency. In addition there is a suggestion that the relationship is becoming flatter at the high frequency end. Thus vibrational susceptibility is not independent of frequency.

Summary

The characteristic of a vibration determining its effect on fresh concrete is the peak velocity and it is linearly related to the fluidity under vibration. The proportionality constant, termed the vibrational susceptibility, is related to the workability of the concrete and is greatest at low frequency and declines as frequency increases.

PRACTICAL IMPLICATIONS

Frequencies between 50 and 200 Hz are in common use for vibrators [4]. Table and surface vibrators operate towards the lower end, formwork and internal vibrators towards the top end. In contrast, Figure 3 shows that the vibrational susceptibility at 16–30 Hz is about four times that at 200 Hz. Thus, in principle, for a given mix and flow geometry the fluidity and hence the rate of flow at low frequency could be four times that at high frequency, provided the same peak velocity can be achieved. This could mean a fourfold increase in the throughput of concrete in the vibratory compaction stage of concrete placement. In practice, because peak velocity is the product of amplitude and frequency it is only possible to achieve high velocity at low frequencies if the amplitude is very high and therefore amplitude is likely to be the limiting factor in any equipment. The significant effect on site productivity implied by the greater throughput could be realised if concrete technologists and equipment manufacturers work together to carry out vibratory compaction at low frequencies in equipment with maximum amplitude.

CONCLUSIONS

Application of vibration to fresh concrete removes its yield value and permits it to flow under its own weight. The fluidity of the vibrated concrete is directly proportional to the peak velocity of the vibration where the constant of proportionality, the vibrational susceptibility, is a characteristic of the material which decreases as frequency increases. Vibration equipment should therefore be operated at the combination of lowest frequency and greatest amplitude possible in order to maximise the fluidity of the vibrated concrete.

ACKNOWLEDGEMENTS

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THE PLANNING AND DESIGN OF CONCRETE MIXES FOR TRANSPORTING, PLACING AND FINISHING

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ABSTRACT. The paper discusses the properties of concrete appropriate to a range of construction methods, operations and environments and how these properties can be achieved in practice. The procurement and production of concrete is an integrated process involving the designer, the construction planner and the concrete producer. Similarly, the specification of concrete must also be integrated between the parties. In practice, the designer begins the process by defining requirements and the construction planner, in conjunction with the producer, completes the specification, taking account of construction methods, environmental conditions and economic use of materials.

Keywords: Concrete mixes, Specification, Methods of construction, Planning, Transport, Placing, Finishing, Mix design process, Workability, Stiffening time.

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INTRODUCTION

Background

The report supersedes CIRIA Report 17, Handling and Transporting Concrete: Time and Temperature Limitations [1] and is one of a series of state-of-the art guides dealing with good practice in concrete construction in the U.K. and other countries with temperate climates.

Since the publication of CIRIA Report 17 there have been significant developments in construction techniques and in the constituent materials available for use in concrete. These have given designers, contractors and concrete suppliers greater flexibility to match concrete mix designs with the particular requirements of modern transporting and placing methods in a wide range of locations and ambient conditions.

Concrete has a proven track record as a successful material in the construction of the highest towers, the longest tunnels, the deepest shafts and the largest of manmade structures and in conditions as extreme as the Arctic tundra and Middle Eastern desert. With adequate planning in all stages of the construction process there are now few conceivable limitations on the use of concrete.

The report provides guidance to designers, specifiers, planners, concrete suppliers and contractors to help ensure that the whole concreting process in planned and implemented in such a way that the required quality is achieved. Guidance is provided on the fresh state Theological properties required of concrete for various operational conditions, and how these properties can be controlled and measured.

Scope

The report provides guidance on the properties on concrete appropriate to a range of construction methods, operations and environments and on how these properties can be achieved in practice. Much of the advice will be relevant to other countries, particularly those with temperate climates, but in more extreme climates specialist references should be consulted[2], or advice sought.

Familiarity with the contents of BS 5328: Parts 1, 2, 3 and 4[3–6] is desirable.

Specialist techniques and applications such as roller compacted concrete [7], underwater concreting [8], road paving [9][10], slip-forming, microsilica concrete [11] and sprayed concrete[12] are not dealt with in the report.

FACTORS WHICH INFLUENCE PROCESS

Design

The procurement and production of concrete which is appropriate for requirements is an integrated process involving the designer, the construction planner and the concrete producer, see Figure 1. Similarly, the specification of concrete must also be integrated between the parties. In practice, the designer begins the process by defining requirements and the construction planner, in conjunction with the producer, completes the specification, taking account of construction methods, environmental conditions and economic use of materials.

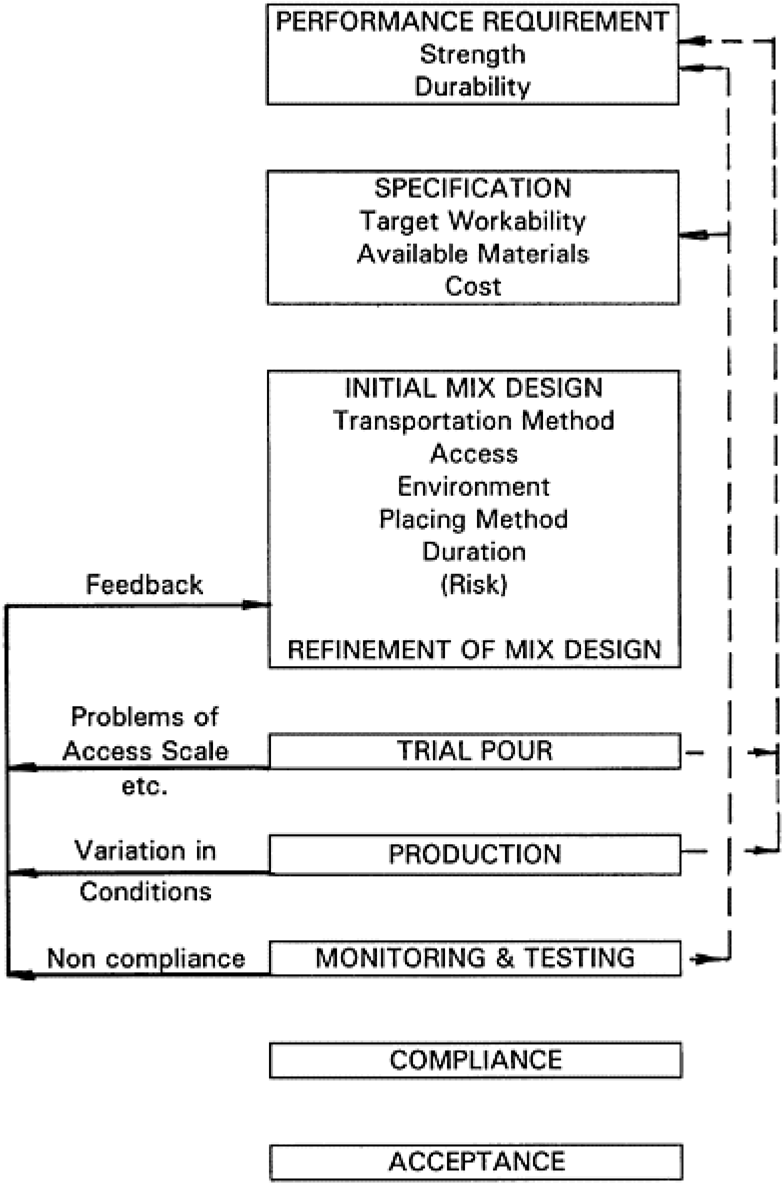

Figure 1: The Process

Approaches To The Specification

Specifications may be written in many ways. The two extremes are: a pure performance specification where performance requirements for the concrete, such as strength, are set out, together with the means of assessing them; alternatively, a prescriptive approach sets out the precise requirements for the class and proportions of all materials to be used. The prescriptive approach may even extend to the specification of maximum pour size, location of construction joints and limitations on construction sequence.

In the future, concrete specifications may contain durability performance requirements such as effective diffusion coefficients, carbonation criteria or permeability. At present there are no agreed tests, criteria or procedures for testing the above parameters. However this is an active area of research, and developments in this field should be encouraged.

In practice, the majority of specifications at present are combinations of performance and prescription.

Whichever approach is adopted, the designer must be aware of the implications of his specification on the construction process. The contractor and concrete supplier should not be unduly restricted in their choice of appropriate and economic construction methods and concrete mix designs.

The designer must avoid specifying parameters or limits which cannot be monitored and checked for compliance. Acceptance criteria, coupled with tolerances, must be clearly defined and linked to method of sampling and of testing. The specification should also be clear as to actions required in the event of non-compliance.

Method of Construction

Experience or advice should be used to anticipate the likely method of construction and take due account in design, making it clear in the specification if any assumptions have been made with affect the permanent works. In some cases, the imposition of constraints on the construction process may be unavoidable.

The designer should however be receptive to alternative proposals by contractors and be prepared to consider the implications of methods which may not have been anticipated at the design stage.

Environmental Conditions

The designer should be aware that changes in environmental conditions affect the behaviour of fresh concrete during transporting and placing and that corresponding refinements in mix design are desirable to achieve consistency in properties at the point of discharge into the formwork.

COMPLIANCE TESTING

Monitoring and Testing Regime

The basic principle of a good specification is "that which cannot be measured should not be specified". It is also necessary to be clear and unequivocal about the nature of testing and compliance requirements.

In specifying concrete, the designer must not specify performance limits without defining as a minimum:

- the method and pattern of sampling
- the frequency of sampling
- the method of testing
- compliance criteria
- tolerance on test results

Guidance should also be given on actions to be taken in the event of noncompliance.

Action in Event of Non-compliance

Even when concrete is produced, transported and placed in accordance with good practice and following all necessary controls, there is still a risk of an individual test result failing to comply with requirements. It is likely that difference of opinion on the appropriate course of action following a non-compliance is the commonest source of dispute between designers, contractors and producers. Much of this could be resolved by having a clear procedure for resolution of uncertainty included in the specifications. The steps to be taken should be clearly laid out and should include:

- validate sampling procedure adopted
- validate testing procedure adopted
- establish location and extent of non-compliance
- consider degree of non-compliance and influence on strength and durability
- consider scope and consequences of remedial action
- implement measures to prevent recurrence

Non-compliance with the specified requirements may not be limited to test results. An example might be the occurrence of plastic settlement or shrinkage cracking in slabs, or blemishes or colour variations in formed surfaces.

CONSTRUCTION PLANNING

The designer begins the specification process for concrete by defining design requirements. The construction planner completes the specification by taking due consideration of the chosen construction methods, plant, personnel, the site location and environmental conditions.

The planner must ensure that plant in the supply chain of production—transport—placing—finishing—curing, is matched to the supply rate necessary to achieve continuity of placing with minimal risk of cold joints.

In all concreting operations continuity of supply at a rate to match the capacity of other elements in the supply chain is essential to success. Communication with the concrete producer is essential, with advance notice being given, particularly for larger pours or those likely to exceed normal working hours. Some 90% of in-situ concrete in UK is supplied ready-mixed. On-site batching is unlikely to be economic for volumes of less than 20,000 m^3 and production rates less than 200 m^3 per day [14]. There may be circumstances, e.g. where access is unusually difficult, where onsite batching of smaller production volumes and rates is economically feasible.

Truck mixers are the usual delivery vehicle for normal concretes. Truck mixers provide the facility for keeping the concrete agitated during long haul distances. They also allow remixing of the fresh concrete with superplasticisers added on site. In many situations it is possible to position the truck alongside the work and discharge directly into place, for which a concrete of high workability would be appropriate. There must be clear access that is firm enough to support a truck weighing, generally about 24 tonnes. Particular attention should be paid to temporary works design when truck mixers are required to be parked close to the edge of an excavation. Where the concrete is required to be placed above ground level it may be discharged into skips and handled by crane. Alternatively, the concrete can be discharged directly into the hopper of a mobile concrete pump. The most common truck mixer used in the UK holds $6m^3$ of concrete, although $4m^3$ truck mixes are available in some areas and may be useful where access is restricted.

Placing by crane and skip enables concrete to be placed at the lowest practical slump consistent with compaction by vibration, and remains a popular technique. Typical rates of placing are $8–12^3$/hour. Selection of the appropriate crane to provide safe access to all parts of the pour is essential.

In contrast to the low rates of placing concrete using a crane and skip, placing rates of $50m^3$/hour are regularly achieved by pumping, and rates up to 100 m^3/hour are possible[13]. Pumping has the following advantages:

- removes the need for staging, hoists, dumpers, skips etc
- overcomes access problems
- fast, therefore can be cost effective
- pour rates can be maintained at heights

Pumping is one of the most efficient ways of moving concrete. Most pumps can be continuously fed from a truck mixer with a load of $6m^3$ (or even two together), and the concrete pumped in 5–6 minutes.

The high rates of placement require careful planning to ensure adequate resources of operatives and plant are available to compact and finish the concrete.

Other methods of placing include chutes, conveyors, drop pipe or trunking and tremie placing.

The Report considers several other planning aspects including compaction, access, duration or environmental constraints and finishing.

FACTORS INFLUENCING CONCRETE MIX DESIGN

The key to successful mix design is to comply with all specified criteria while attaining a workable concrete which satisfies the job-specific requirements of transporting, placing and finishing. The skill of the mix designer is in achieving these requirements in the most economic way making use of the range of materials at his disposal.

The Mix Design Process

The mix design process begins with the designer's specification which the contractor, as purchaser of the concrete, reviews and selects the appropriate method of specifying the required characteristics of the concrete to the producer. BS5328 permits four methods:

• designed mix
• prescribed mix
• standard mix
• designated mix

In all cases, the purchaser must provide additional information to the producer, most notably the required workability of the mix. In the case of prescribed and standard mixes the purchaser also takes responsibility for selection of mix proportions. Most structural quality concrete will be specified as either designed or designated mixes for which the producer is responsible for selecting mix proportions.

The mix designer may than prepare an initial design based on general principles or past experience of concrete produced to a similar specification and known to have performed successfully. At this stage the mix designer and contractor will make their initial choices based on availability and cost of constituent materials.

The design process should, however, continue to be refined and take account of the method of transporting, placing and finishing, access to the pour location, the environmental conditions, and the planned duration.

Since the refinement process takes account of a range of conditions, each of which may change during the course of a project (particularly environmental conditions), it is important to appreciate that variations in mix design should be admissible, indeed encouraged, to respond to these changes. The mix design should therefore be regularly reviewed throughout a project. To achieve this match of properties with varying conditions requires communication and co-operation between all parties involved.

Changes may of course be essential in the event of any non-compliance during monitoring and testing. The Process is illustrated diagrammatically in Figure 1.

Characteristics to be considered

In addition to the designer's specified parameters, the key properties to be selected and controlled for efficient concrete construction are:

• workability
• stiffening time
• cohesiveness

The required workability and cohesiveness at the time of placing are related to the nature of the pour and the placing equipment. Initial workability and cohesiveness may require to be modified to suit the method of transporting, the ambient conditions and the transit times of the fresh concrete. The required stiffening time must take account of the planned time between successive lifts, and the effect on formwork pressures and finishing methods.

All properties are likely to be affected by environmental conditions. Other characteristics such as heat evolution are particularly important in large volume pours and are discussed in CIRIA Report 135[13].

Section 4.3 of the Report discusses the implications of transporting, placing, access duration and environment, and finishing methods on the desired characteristics of the fresh concrete, providing guidelines for pumping both normal and lightweight concrete, tremie placing, and for varying ambient conditions. Measures to reduce bleeding, crazing and cracking are also suggested.

Section 4.4 of the Report discusses the influences of mix constituents on the fresh concrete characteristics and describes ways in which these can be tailored to match specific requirements and environmental conditions. The approach adopted is to consider the effect of changes in materials or conditions from an initial ‘reference’ condition.

The anticipation of all potential variations in factors which influence the behaviour of fresh concrete is, however, not generally necessary. Concrete is a tolerant material and in its fresh state a particular mix will perform adequately over a range of varying influences. However, its performance can be improved if the major influences are anticipated and appropriate refinements made to mix design.

Workability

A primary influence on initial workability is water content. Given that strength is likely to be a basic governing requirement and that there is a correlation between strength and water/cement ratio, changes in water content will require corresponding changes in cement content. The initial mix design process will be based on a resolution of strength and workability requirements based on this relationship.

The following are discussed in detail in this section of the Report:

- water and retempering
- cement and blended cements
- sand
- admixtures and their effect on workability
- temperature
- cohesiveness
- stiffening time and control with admixtures.

Table 1: Factors affecting changes to the water requirement of concrete constant slump (after Owens)

Consideration	**Reference Condition**	**Condition change from Reference**	**Change of water requirement (Litres/m^3)**	
			More	**Less**
Cement	Portland cement to BS12 (PC 42.5)	PC 52.5(to BS12)	5	
		SRPC to (BS 4027)		5
		PC with 30% pfa to BS 3892: Part 1		15*
		PC with 50% pfa to BS 3892: Part 1		30*
		PC with 60% ggbs to BS 6699		10*
Aggregate				
		10mm	35	
(a) Size	20mm nominal size to BS 882	15mm	15	
		30mm		15
		40mm		25
(b) Shape	Irregular	Angular	10	
		Rounded		10
(c) Grading	Continuous	Gap		10
		Oversanded	10	
Admixture	Plain concrete - no admixture	Water reducing* Single dose		10
		Double dose		20
		Set retardation 24h		15
		Air-entrainment 3–5%		10
		5–8%		20
Concrete Density	2250–2500 kg/m^3	1750–2000 kg/m^3	20	
		2000–2250 kg/m^3	10	
		2500–2750 kg/m^3		10
Concrete	20°C	Reduced to 2°C		15

Temperature		10°C		10
		Increased to 25°C	5	
		30°C	15	
		40°C	30	
Period of agitation after mixing	Up to ½h after adding water	Increased to between: ½–1½h	5	
		1½–2½h	10	
		2½–3h	15	
		3–3½h	20	
*Note: pfa and ggbs may have no, or very little, water reducing properties if water reducing admixtures are also used.				

There are many influences on the workability of fresh concrete, each of which can potentially be used to refine the characteristics of concrete as requirements, or ambient conditions, change. There is a shortage of data to allow firm guidelines to be given to enable mix designers to quantify these refinements, and the many interacting variables and changes in local conditions and site-specific requirements may make it difficult for anything other than general guidelines to be given. With this caveat, Table 1 gives a summary of the main factors affecting changes to the water requirement of concrete for constant slump, and provides indicative adjustments in water quantity to a notional "reference" condition.

CONCLUDING REMARKS

The key to successful transporting, placing and finishing of concrete is to:

- plan the operation
- adopt quality assured procedures
- define the characteristics required of concrete throughout the operation
- carry out an initial mix design
- refine the mix to match characteristics to those required
- continuously review conditions and make further refinements in response to any changes

General indicators to assist mix designers in refining initial mix designs are given.

Section 5 of the Report contains a number of case studies to illustrate the application of these principles.

Case Study 4—Air-entrained concrete in bridge deck

In this example a higher strength grade of C40 is combined with a designer's requirement to have air-entrainment. The pour is to take place in winter, with a long haul time and is to be pumped into place. The mix designer adopts a composite cement with ggbs to provide better control of air. See Tables 2 and 3.

The problem, essentially, is how to sustain the 4.5% entrained air for such a long haul time, particularly under winter conditions and originally with such a high cement content (550 kg/m^3). Reducing the cement content to 440 kg/m^3 helps, together with the more angular shape of the granite coarse aggregate and the sharper nature of the sand. The cohesiveness of the mix will be improved by the air-entrainment, but while the cement water paste would have good cohesiveness, the sand proportion has to be raised to produce sufficient mortar to give the mix a pumpable quality. It is the combination of reducing the cement and water contents, balanced by raising the sand content, leading in turn to a reduction in coarse aggregate that helps reduce what is potentially a very sticky mix. Care needs to be taken with the trial mixes and in monitoring the effect of mix temperature on the amount of air entrained.

Table 2: Case Study 4 Air-entrained Concrete in Bridge Deck

Preliminary Mix—Design Limitations		**kg/m^3**
Cement pfa ggbs	PC 42.5	550
Aggregate	Nominal max. size 20mm	
Sand	BS882	425
Coarse	BS882	1,095
Admixture	–	–
Water (Free)		210
Density (Fresh)		2,280

Comments	**General Considerations**	**Special Considerations**
Strength, N/mm^2	50 (modified to 66 for AE)	
w/c Ratio	0.40	
Workability	75 mm slump	
Cement	Use ggbs to control air.	Limit to 40% because of extended setting time in Winter.
Aggregates		
Sand	55% passing 600μm gravel	
Coarse	Crushed granite	
Admixture	Separate air-entrainer and plastiser to reduce water.	

Other	Provision for water addition on site MUST be made to adjust workability for pumping.	Winter conditions favour long haul time of 45 minutes.

Recommended Mix		**kg/m³**
Cement pfa ggbs	PC 42.5 – BS 6699	265 175
Aggregate Sand Coarse	Nominal max, size 20mm BS882 BS 882—Crushed granite	 650 1,010
Admixture	BS 5075: Parts 1 & 2 air-entrainer and plasticiser	1.6 litres
Water (Free)		175
Density (Fresh)		2,275

Table 3: Case Study 4 Air-entrained Concrete in Bridge Deck

Design Limitations								
Strength Grade	Nominal Maximum Size of Aggregate (mm)	Aggregate Type	Sulphate Class	Cement Type	Minimum Cement Content (Kg/m³)	Maximum Free w/c Ratio	Density	Other
C40	20	N/A	1	N/A	325	0.55	Normal	4.5% Airentrainment

General Considerations										
Ready Mix or Site Mix	Site Tran spor t	Type of Elem ent & Thick ness	Reinfor cement congestio n	Size of Pour (m³)	Acces s to Form work	Climatic Cond ition s	Speed of Const ruction	Com pactio n M ethod	Finishing Consid erations	Frame work Comp lexity
Ready Mix	Pump	Bridge Deck	Congested	45	From Top	North European Winter	Normal	Internal Vibration	Wood Float	Intricate

Special Considerations							
Deep Lifts	Large Volume Pours	Hot & Dry Conditions	Severe Winter Conditions	Long Hall	Top Forming	Slip Forming	Early Striking Time
No	No	No	Yes	45	No	No	No

				Minutes			

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ROLLER COMPACTED CONCRETE: DEVELOPMENT AND USE IN QUEENSLAND, AUSTRALIA

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ABSTRACT. Roller Compacted Concrete development in Queensland has involved a mix of laboratory experimentation and practical experience. The optimum grading for RCC aggregate and optimum placement moisture content has been established. Continuous mixer performance has been evaluated by analysing sample cylinder strength tests. An accurate cement content test has been developed and testing of the moisture content of the RCC mixture has been evaluated. A laboratory trial has shown that the desirable moisture content of the mix, as established in the field, coincides with the moisture content giving the highest strength and that strength varies significantly with density. Construction control testing of cement content, moisture content and pavement density ensures pavement uniformity and this is verified by simply prepared concrete cylinders.

Keywords: Aggregate Grading, Cement Content, Moisture, Density, Cylinder Strength.

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INTRODUCTION

Roller Compacted Concrete pavement construction in Queensland, Australia, undertaken by the Author commenced in 1987. The interest in RCC arose out of the widespread use in Queensland of cement treated base (CTB) and subbase layers in new pavements, developed by the Queensland Department of Transport. These cement treated materials use high quality aggregates, are mixed in continuous mixing plants and placed using heavy duty paving machines.

At this time the construction of RCC pavements was being undertaken elsewhere in Australia (1) and RCC was being used for dam construction (2). A review of published material from other sources revealed that no consensus had been achieved as to mix design, testing procedures and construction techniques, particularly in relation to RCC used in pavements. The Author therefore decided to apply his experience with CTB pavements to independently developing appropriate techniques for RCC pavements. To date 16 projects or project stages have been undertaken in Queensland and these have enabled most aspects of RCC pavement design and construction to be investigated and satisfactorily resolved.

RCC AS A PAVEMENT MATERIAL

Properties of Hardened Concrete

RCC with a compressive strength equivalent to conventional concrete can be readily prepared but properties such as flexural strength and fatigue resistance are more important parameters in determining pavement performance. Studies have concluded that the fatigue resistance of RCC is equal to that of conventional concrete (3) and that the compressive strength/flexural strength relationships are similar (4). Long term strength results comparing RCC to conventional concrete indicate that RCC having a similar strength to a conventional concrete at 28 days has greater long term strength gain (5). The conventional concrete design method contained in the NAASRA Pavement Design Guide (6), whereby design is based on a flexural strength derived from the 28 day cylinder compressive strength, is therefore appropriate.

Properties of Fresh RCC

In order for RCC to be placed with equipment used for granular pavements the fresh material must behave in an essentially noncohesive manner. Correctly proportioned RCC therefore exhibits all of the characteristics of a gravel and is controlled and tested accordingly. The laboratory compaction curve of moisture content versus maximum dry density is prepared and concepts such as field compaction measured as a Relative Dry Density and moisture content (weight water/weight dry materials) as referenced to Optimum Moisture Content apply.

RCC MATERIALS

Cementitious Material

RCC mix designs having flyash contents varying from none (7) to 60% (8) have been reported. Cement Type GB (Flyash Blend), composed of 75% Ordinary Portland Cement and 25% Flyash, is readily available in Queensland at a cost about 10% below that of Portland Cement. Because the use of flyash has the beneficial effect of reducing early thermal rise and consequent shrinkage Type GB cement is normally used in Queensland for this, and economic, reasons.

Aggregates

The Queensland Department of Transport Specification MRS11.05 (9) covers aggregate for granular and CTB pavements. The highest quality fine crushed rock, Type 1.1, is required to meet stringent aggregate strength and stability requirements, and is produced by quarries in parallel with concrete aggregates.

The grading of Type 1 aggregates has been arrived at as being that required to maximise the density of the compacted material i.e. it is the grading that allows progressively smaller particles to fill the voids left by the larger particles. This grading and that of a typical combined grading curve for conventional concrete aggregates is shown in Table 1. Type 1 is finer on all sieves and contains a significant proportion of material finer than 75 microns.

Since 1987 Type 1.1 aggregates from a number of quarries have been tested to determine concrete strength and significant differences have been recorded. As the soundness of the aggregates are similar, and certainly high enough not to be a factor in determining compressive strength, some characteristics of the fines is the variable. Aggregate with a grading closer to conventional concrete aggregate has been trialed and this grading is also shown in Table 1.

Table 1 Grading of RCC and Concrete Aggregates

Sieve Size mm	Type 1	Percentage Passing Brisbane 40MPa	Trial Blend
26	100	100	100
19	87	100	99
9.5	69	58	68
4.75	54	37	42
2.36	39	32	27
0.425	18	10	11
0.075	7	0	4

This trial however established that aggregate grading has a strong influence on fresh material behaviour, the trialed material exhibiting a “stickiness” that made it difficult to place. Therefore the suitability of the grading of RCC aggregate is determined by a need for the grading to be such that the cement paste is sufficiently dispersed over the surfaces of the various particles in the mix for the non-cohesive characteristic to be retained. The Type 1 grading, by coincidence, appears to be close to ideal.

It should be noted that in Type 1.1 aggregate the presence of clay fine particles is acceptable provided that the plasticity index of the fraction passing 0.425mm is less that

4. However, conventional concrete experience has shown that clay fines are detrimental to strength. The desirable material for fine particles appears therefore to be “rock flour”, material produced only by the crushing of a stone meeting the quality requirement of the course aggregates.

MIXING OF RCC

The Aran continuous mixer is used extensively for both CTB and granular pavement work in Australia. The earlier model, the ASR200, is readily available and has been used on most RCC projects. Although later model machines are very sophisticated, incorporating belt weighers and recording electronics, the performance of the machine is best appreciated by realising that it essentially volumetric in its operation, as the feed rates of aggregate and cement are determined by the opening size and belt speed of the respective feeders.

The detention time of the material in the mixing chamber is low (about 10–20 seconds) and hence the efficiency and uniformity of mixing needs to be verified. Two examples of the performance are given in Table 2, the first being for an RCC project and the second being a performance trial carried out for high strength, zero slump, concrete. The results in Table 2 clearly indicate that the Aran pugmill produces consistent mix and hence any variations that may occur in a project are not caused by deficiencies in mixing.

Table 2 Uniformity of Mixing of Aran Continuous Mixer.

Model	No. of Samples (1 Pair Cylinders)	Mean Mpa	Standard Deviation
ASR200	8	32.4	1.64
ASR250X	4	65.25	2.01

CEMENT CONTENT DETERMINATION

Pugmills can be calibrated only approximately by means of setting the volumetric feed rate of the cement and aggregate. There is the need therefore to have an accurate test to monitor the actual cement content of the mix. The Queensland Department of Transport has for many years used the Cement Content of Cement Treated Material (Heat of Neutralisation) Test Method Q116B-1978 (10) for this purpose. The test method determines the cement content by measuring the heat rise caused by the reaction between the cement in a sample of mixed material and a “buffer” solution of a blend of sodium acetate and acetic acid. A curve of cement content versus temperature rise is prepared in the laboratory using mixes with known cement contents, and this curve is then used in the field to monitor the work.

There have been some variations in results reported using this method. However, in the absence of a practical alternative, this has been developed for RCC use. Because of the higher cement content of RCC as compared to CTB, more buffer is required to ensure that there is sufficient for the reaction to consume all of the cement. It has been established that the original Department of Transport buffer composition, being 150 g/litre of sodium acetate and 240 g/litre of acetic acid is suitable. A number of different ratios of buffer and mix have been investigated with the aim of ensuring that the test is accurate, while minimising the cost of testing, the chemicals being rather expensive.

The proportions that allow cement contents of between 10% and 20% (weight cementitious/weight dry aggregate) to be accurately measured has been established at:

RCC Mixture	2 Kg
Buffer	3 litres

In the Department of Transport Test Method the initial temperature is taken as the mean of the buffer and mixture temperatures. However this does not take into account the specific heat of the components, which can be considered as follows:

$T_m (M_g SH_g + M_S SH_S) = T_g M_g SH_g + T_s M_s SH_s$

Where T_m=mean temperature of combined material

T_g=RCC mixture temperature

T_s=Buffer temperature

M_g=Mass of RCC mixture

M_s=Mass of Buffer

Sh_g=Specific heat of RCC

Sh_s=Specific heat of Buffer

The Specific Heat of concrete is 970–1050J/Kg. °K and water is 4187J/Kg. °K. Assuming the buffer has a specific heat 4 times that of the RCC and for a ratio of 2Kg RCC to 3L buffer, this equation reduces to:

$T_m = T_s + 0.14(T_g - T_s)$

This equation has been confirmed experimentally in the laboratory by adding gravel to buffer at varying temperatures and recording the stabilised temperature of the mixture, with the constant found to be 0.15.

A typical laboratory prepared calibration curve is shown in Figure 1. Test points with widely differing gravel and buffer temperatures are shown in Table 3, which clearly demonstrates the accuracy of the test method. These results, and experience from preparing calibration curves for a number of projects, has confirmed that the test is accurate to within 0.5% cement content. This represents about 10Kg of cementitious material per cubic meter of compacted RCC which is certainly sufficiently accurate for practical purposes.

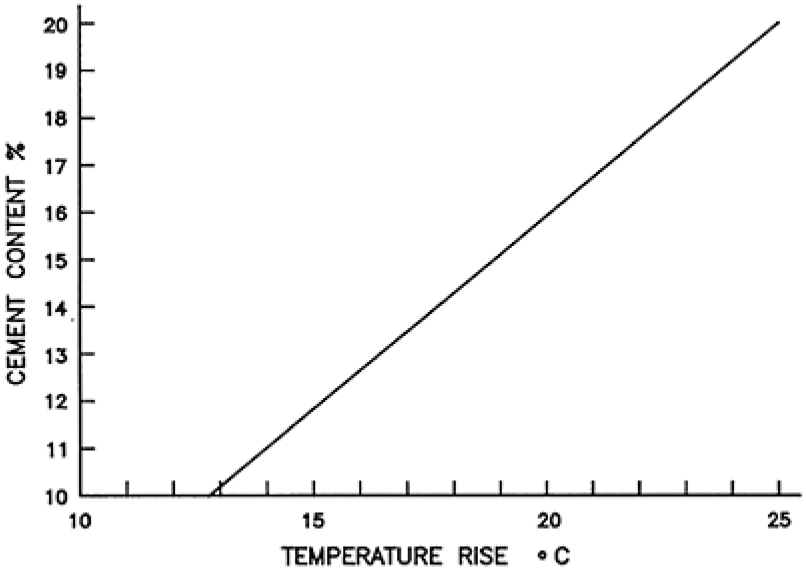

Figure 1 Cement Content Calibration Curve

Table 3 Laboratory Testing of Cement Content

Temperature (°C)					Calculated Cement Content	Actual Cement Content
Initial RCC	Initial Buffer	Calculated Combined	Recorded Final	Temp. Rise		
19.0	30.9	29.1	45.7	16.6	13.2	13.0
43.4	29.2	31.3	47.3	16.0	12.8	13.0
23.6	17.8	18.7	39.6	20.9	16.7	16.5
37.1	39.0	38.7	59.4	20.7	16.6	16.5

STRENGTH/MOISTORE/DENSITY RELATIONSHIP

Concrete cylinders for laboratory trials and construction control purposes are prepared by compacting RCC into 300mm high by 150mm diameter concrete cylinders using a Standard Compaction hammer and 100% Standard compactive effort (5 layers 80 blows/layer). Standard Compaction has been selected as it accords with general Department of Transport practice and makes the process easier to perform.

It has been established that the best placement moisture content is between 1% to 1.5% below OMC Standard. A laboratory trial to determine the effect of varying moisture contents and compaction levels on strength has been conducted and the results are presented in Figure 2. It can be seen from Figure 2 that the desirable placement moisture content coincides with that which gives optimum strength, and that strength is sensitive to density.

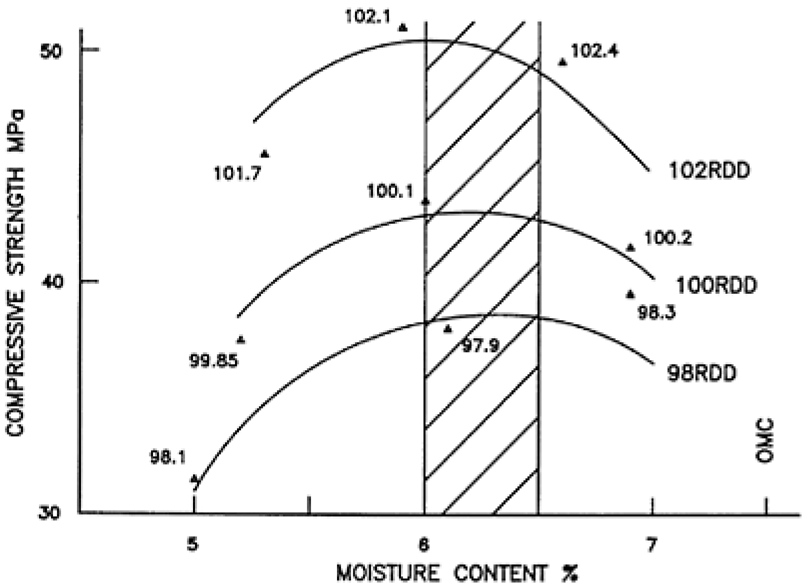

Figure 2 Strength/Moisture Content/Density

MOISTURE CONTENT

Because the accurate measurement of the moisture content of the mix is critical to controlling both water addition and measuring pavement density (expressed as a Relative Dry Density) it needs to be determined how the hydration of the cement affects the apparent moisture content. Department of Transport practice for CTB work is to define time limits within which tests must be conducted. In RCC work in Queensland the use of retarder has been found to be necessary to allow finishing to be completed prior to initial set. Therefore the effect of retarder on material behaviour and apparent moisture content has been investigated.

Laboratory moisture content testing has been carried out with samples incorporating retarder. These tests have shown that retarder is effective in preventing apparent moisture loss due to cement hydration for a period of at least 4 hours from mixing. Using a microwave oven very good correlations between added and calculated moisture contents have been obtained. Further, it has been established that a set retarding admixture (not a water reducing admixture) has no affect on the MDD/OMC curve and the behaviour of the material in the field is unaffected.

CONSTRUCTION

Moisture Control

It is necessary to maintain the moisture content within about +/−0.5% of the target moisture content. The moisture content in the aggregate stockpile may contain a significant proportion of the total requirements and can easily vary by more than the desirable range for the mixed RCC. Although, with experience, it is possible to manually assess the fresh RCC moisture content reasonably accurately it has proved difficult to keep within the desirable range when the aggregate stockpile moisture content varies. By premixing a variable stockpile prior to loading into the pugmill the need for continual adjustments to the amount of added water can be eliminated.

Cement Content

Prior to commencing work on site (or when a new batch of buffer is prepared) a "check point" on the calibration curve is obtained on site by testing a mixture prepared from known weights of on-site materials. This provides fail safe verification of the test procedure. The test takes about 15 minutes to perform and because of the direct relationship between volumetric and mass feed rates in a pugmill only one test is required to establish the correct ratio of settings between cement and aggregate and one further test to confirm the cement content after adjustment of the settings. Further cement content testing is carried out throughout production as part of a Quality Assurance program.

Density

Department of Transport practice is to accept a particular lot of work in accordance with a Characteristic Value formula which in effect requires all results to be above the nominated level. Sand replacement density tests (with microwave oven drying) or nuclear densometer testing gives rapid test results, enabling rolling to be adjusted as necessary so that the minimum density of 100% RDD Standard is achieved.

Concrete Cylinders

Cylinders are prepared on site from fresh mix. Because the moisture content is below Standard Optimum, densities are marginally below 100% RDD and hence cylinder strengths are below the in-situ strengths. This is therefore a very simple and conservative means of strength verification. Provided the sample is protected from drying out during cylinder preparation minimal density and strength differences occur between the two cylinders in a sample.

CONCLUSIONS

1. The optimum grading for RCC aggregates has been established.

2. The reason for strength variation between different sources needs further investigation.
3. An accurate method for determining the cement content of the fresh mix has been developed.
4. The desirable moisture content for placement and that for maximum strength coincide.
5. During construction simple techniques and tests are used to control and verify the quality of the finished pavement.

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