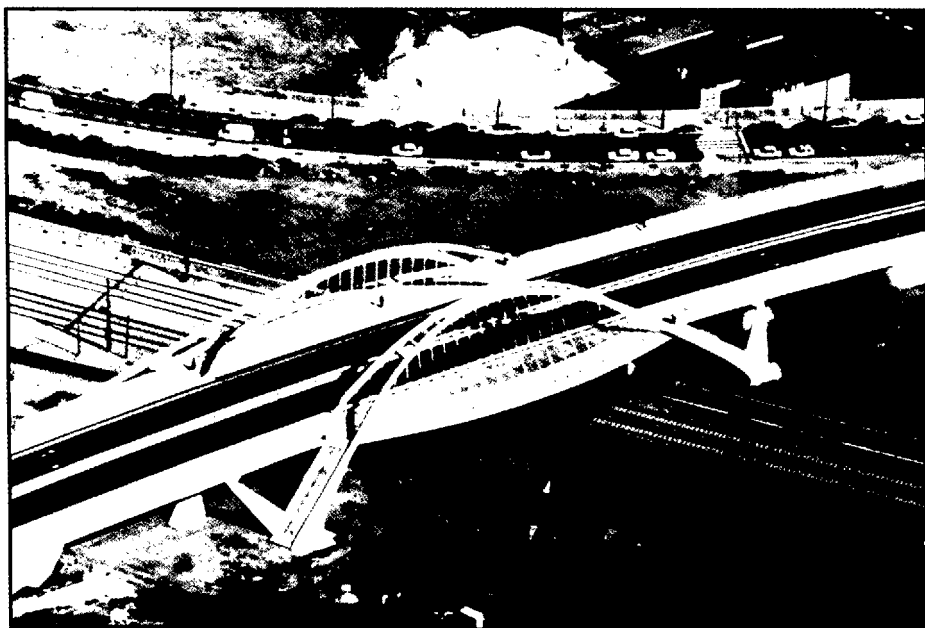


2000 CANMET/ACI

INTERNATIONAL • CONFERENCE • INTERNATIONALE



**DURABILITY
OF
CONCRETE**

**DURABILITÉ
DU
BÉTON**

EDITOR • V. M. MALHOTRA • RÉDACTEUR EN CHEF

Durability of Concrete

Proceedings Fifth International Conference Barcelona, Spain, 2000

Volume I

Editor

V. M. Malhotra



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PREFACE

The Canada Centre for Mineral and Energy Technology (CANMET) of Natural Resources, Canada, Ottawa, has played a significant role in Canada for over 35 years in the area of durability of concrete. In August 1991, CANMET, in association with the American Concrete Institute (ACI), and the Institute for Research in Construction/National Research Council, Ottawa, sponsored the Second CANMET/ACI International Conference on Durability of Concrete. The conference was held August 4-9, 1991, in Montreal, Canada, and was a follow-up to the Katharine and Bryant Mather International Conference on Concrete Durability held in Atlanta, Georgia, April 27-May 1, 1987. The refereed proceedings of the Atlanta and Montreal conferences were published as ACI SP-100, and ACI SP-126, respectively.

In 1994, CANMET, in association with ACI and several other organizations in Canada and France, sponsored a third international conference on concrete durability. The conference was held May 22-28, 1994, in Nice, France, and the conference proceedings, consisting of 65 refereed papers, were published as ACI SP-145.

In 1997, CANMET, in association with ACI and several other organizations in Australia, sponsored a fourth international conference on the subject. The conference was held August 17-22, 1997, in Sydney, Australia. The two-volume proceedings of the conference consisted of 81 refereed papers, and was published as ACI SP-170.

In 2000, CANMET, in association with ACI, the Japan Concrete Institute, and several other organizations in Spain and Canada, sponsored a fifth international conference held June 4-9, 2000, in Barcelona, Spain. More than 120 papers from 35 countries were received and peer reviewed in accordance with the policies of the American Concrete Institute; 73 were accepted for publication. The accepted papers deal with all aspects of concrete durability.

In addition, several sessions dealing with sulfate attack, superplasticizers and supplementary cementing materials, and near surface testing for the durability of concrete were organized. To all those whose submissions could not be included in the symposium volume, the Institute and conference Organizing Committee extend their appreciation for their efforts and interest.

In addition to the papers that have been published in the refereed proceedings, more than 30 other papers were presented at the conference. A number of these were published as supplementary papers, and the authors of these papers are encouraged to publish in technical journals of their choice.

Thanks are extended to more than 20 members of the review panel that met in Torremolinos, Spain, in May 1999 to review the papers. Several other people also assisted in the review process, and their help is gratefully acknowledged. Without the dedicated efforts of the reviewers, it would not have been possible to have the proceedings ready for distribution at the conference. The cooperation of the authors in accepting reviewers' suggestions, and in revising their manuscripts accordingly is greatly appreciated. The authors are also to be commended for their prompt return of the revised manuscripts.

The help and assistance of A. Bilodeau (Chairman), B. Fournier, and R. Chevrier (members) of the slide review panel are gratefully acknowledged. Thanks are also extended to H. S. Wilson, M. Venturino, and G. D. Brearly for their help in processing the manuscripts. The contributions of the ACI staff are also recognized.

As an integral part of the conference, a special dinner to honor P. Scheil was held to acknowledge his outstanding contributions to concrete durability over the last 25 years.

V. M. Malhotra
Editor

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SP 192-1

Ordinary and Long-Term Durability of Reinforced Concrete Structures

by M. Collepardi

Synopsis: Durability of reinforced concrete structures (*RCS*) seems to be poor when compared with those of ancient un-reinforced structures. When ordinary durability (service life of 40-50 years) is needed, the poor behavior of *RCS* stems from human negligence in adopting the well consolidated and available experiential knowledge. However, for long-term durability requirements (service life of 100 years and more) the inherent vulnerability of the steel-concrete system must be taken into account.

The inherent vulnerability of *RCS* substantially depends on the following “weak points” of concrete:

- (i) Low tensile strength
- (ii) High modulus of elasticity
- (iii) Microcracking caused by restrained thermal and drying shrinkage or service loading.

This paper critically examines some possible future scenarios to achieve long-term-durability in *RCS*, including:

- a) Improvement in the corrosion behavior of the metallic reinforcement through the use of corrosion inhibitors, protection of the reinforcement with a coating, change in the composition of reinforcing bars, or cathodic protection.
- b) Use of non metallic reinforcement.
- c) Increase in the tensile strength and/or ductility of concrete mixtures based on rubber-like polymer additions.
- d) Surface coatings for concrete protection.

Keywords: cathodic protection; corrosion; durability; epoxy resins; reinforcement; steels

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INTRODUCTION

Paradoxically, modern concrete structures, with metallic reinforcements, are less durable than the ancient un-reinforced concrete structures.

Some Roman plain concrete structures are still in excellent condition even after 2000 years or more. Those which today appear in form of romantic ruins owe the loss of their integrity more to extra-ordinary traumatic and cumulative events of the past - earthquakes, fires, wars - rather than to degradation processes inherent to the material itself. For the front cover of the Proceedings of the Kumar Mehta Symposium (1) the editors selected a picture of the *Pantheon* dome in Rome, a still sound, intact and majestic building made of concrete with a lime-pozzolan mixture as binder and crushed bricks or natural pumice as lightweight aggregate. On this picture, the editors placed an inscription referring to this concrete: "*But concrete can be durable*".

The long term durability of this ancient cementitious material highlights even more the relatively poor behavior - in terms of durability - of modern reinforced concrete structures (*RCS*), most of which have lost their original serviceability in less than one century, and in some cases in less than a few decades. When referred to modern reinforced concrete the above inscription should be changed into "*Can it be durable?*"

To find possible answers to this question, this paper will address the three aspects of the problem:

- The meaning of durability of modern concrete structures
- The causes of deterioration in *RCS*
- Conceivable scenarios to design and achieve long-term-durability in *RCS*.

WHAT DOES DURABILITY MEAN?

The durability of a reinforced concrete structure is the capability of the structure of maintaining its original **functional and structural characteristics** for the expected service life and exposure conditions it was designed for. It does not necessarily coincide with the durability of concrete, the latter being the capability of the material by itself of keeping the original **properties** for a certain period of time. In fact, the durability of *RCS* depends not only on the

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durability of the concrete, but also on aspects of design (cover thickness, density and positioning of reinforcing bars, and surface protection, if any) and on the **execution techniques** (transport, placement, compaction, and curing of the concrete mixture).

Standards and recommendations are available in Europe (2), America (3), and Japan (4) for the durability of concrete and RCS exposed to various aggressive environments, including humid air, sea water, freezing and thawing, and de-icing agents, when a service life of 40 to 50 years is required. In the present paper this will be termed **ordinary durability** compared with **long-term durability**, which refers to a longer service life, with a minimum of 50 years up to 200 years (5) and even more. Long-term durability is needed, for instance, in infrastructural works of particular social importance which require great investments (e.g., underwater tunnels, long span bridges, highway networks, etc.) or structures of particular architectural interest (e.g., monumental works, churches, state buildings, etc.). Significant examples of the latter structures are the *Grande Arche* in Paris and the *Opera House* in Sydney.

Wondering whether these monumental buildings will last, for instance, for at least 500 years is more than legitimate, if we compare the experienced service life of reinforced concrete buildings in the present century with those of ancient monumental works built in this millennium.

Michelangelo - who is perhaps the most famous world-wide sculptor and painter, but who was also a very fine architect - designed *Palazzo Farnese* in Rome in the 15th century. This building is still in service as the residence of the French Embassy in Italy. Can we hope for a five-century service life in modern reinforced concrete architectural works such as that achieved by many Renaissance buildings in Italy, as well as in other countries in Europe? Although Michelangelo was more an artist than an engineer, he always took into great account the quality of the materials he used to achieve the longest possible durability for his masterpieces - the marble for the *David* statue in Florence, the colors and the mortar substrate for the *Cappella Sistina* in Vatican, as well as the stones, bricks and mortars for the *Palazzo Farnese* or the *Capitolium Square* in Rome.

Is the present experiential knowledge of modern construction materials lower than that available to Michelangelo and to other architects of the past? If not, why are the durability problems of RCS debated in hundreds of papers, seminars, conferences, and books? According to the author of the present paper, the answer to this key-question can be found by examining two fundamental durability problems in RCS:

- a) **Human negligence** in adopting the well consolidated and available experiential knowledge for ordinary-durability RCS.
- b) **Inherent vulnerability** of the steel-concrete system for long-term-durability RCS.

Both of these aspects are discussed in the next section.

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DETERIORATION CAUSES OF RCS

Human negligence: In RCS exposed to aggressive environments, a service life of 40-50 years (ordinary durability) may not be achieved when one or more of the following recommended actions are not performed (Table 1):

- *Selection of adequate materials* in terms of specified cements, sound and well graded aggregates, chemical and mineral admixtures.
- *Proper mixture-proportioning* in terms of water-cement ratio (w/c) and air-void system as needed for strength and durability requirements based on environmental exposures.
- *Adequate structural design* in terms of concrete cover thickness, position and density of reinforcements, form and dimensions of structural elements.
- *Careful execution techniques* in terms of workability and transport-related slump loss of the fresh concrete, placement, compaction and curing of the concrete mixture.

Two types of human negligence may occur. The first type - the most frequent - involves the selection of a high w/c in relation to the aggressiveness of the environment; the absence of an air-entraining agent in structures exposed to freezing and thawing cycles; the selection of a thin cover in reinforced concrete structures exposed to carbonation or chloride penetration; the addition of uncontrolled amounts of mixing water on the job site because of concrete slump loss; and inadequate or lack of curing after demoulding. All these forms of negligence occur due to the gap existing between the available know-how in concrete durability science (6) and the relatively poor knowledge of concrete technology of design-engineers, architects, and especially contractors. Neville (7) has highlighted this gap and ascribed it to the poor attention paid by schools and universities to teaching concrete technology compared with that devoted to structural design. According to Neville "*Inadequate knowledge of factors influencing the behavior of concrete has harmful consequences in the operations of manual and technical staff. This situation exists because learning about concrete is considered almost below the dignity of the person undertaking sophisticated structural calculations*" (7).

The second type of human negligence is related to some aspects of cement production and concrete manufacturing processes which can objectively be managed with some difficulties in the practical field experience (Table 2). One of these aspects deals with the selection of inert aggregates, not prone to alkali-silica-reaction (*ASR*), when concrete must be produced on a large scale - as is usual for real life RCS. So far, reliable and quick tests to detect potential *ASR* in each individual grain of a big batch of aggregate are still not available. The routine use of fly ash and other supplementary cementitious materials remains the best way of preventing *ASR* in concrete structures where there is the risk of

using alkali-reactive aggregates and a low alkali portland cement is not available (8, 9).

Another and more recent aspect of the second type of human negligence (Table 2) deals with the risk of delayed ettringite formation (*DEF*) occurring when high sulfate content clinker is used in the production of modern portland cements (10). According to the available standard tests, only the total sulfate content of cement - and not that of the clinker phase - may be checked. Therefore, the *DEF*-induced concrete distress can not be managed easily. This type of deterioration has been growing, in prestressed concrete structures and particularly in concrete ties, since the last decade. This is due to two concurrent events, besides the exposure to humid environment (Fig. 1): the unwitting increase since the 80s in the sulfate content of the clinker phase related to the use of sulfur-rich wastes and fuels in the kiln, and the increase in microcracking related particularly to the high, uncontrolled and non uniform stress distribution in prestressed and/or steam-cured concrete structures (11). Therefore, this type of human negligence could be eliminated through better control in the clinker production process by cement producers, and in the stress distribution in the *RCS* by design engineers.

Inherent vulnerability of RCS: In contrast to what happens in the laboratory, real structures are subjected to static and dynamic loads. Moreover, the additional deterioration which is observed in real *RCS*, compared with that of the specimens stored in a laboratory, is due to the following "weak points" of concrete (Table 3):

- (i) Low tensile strength.
- (ii) High modulus of elasticity which is responsible for the transformation of restrained thermal or drying shrinkage into relatively high tensile stresses.
- (iii) Microcracks formed as a consequence of (i) and (ii).

These microcracks represent preferential paths for the penetration of aggressive environmental agents - such as air, water, and sulfate, chloride, and alkali ions - through the mechanisms of diffusion and capillary absorption through the cracks. This means that the concrete cover can be penetrated by the aggressive agents independently of the porosity of the cement matrix. This promotes the corrosion of the steel reinforcement, characterized by a disruptive expansion which accompanies the change of the iron into the corresponding oxides (rust). Once this process is initiated, the microcracks in the concrete cover grow into macrocracks and then, after an initial induction period, the duration of which depends on the aggressiveness of the environment, the degradation process increases very rapidly (12).

When ordinary durability is required - as for *RCS* with an expected service life of 40 to 50 years - the presence of microcracks in the concrete cover and their transformation into macrocracks plays, in general, a role of negligible importance provided that all deterioration causes related to human negligences

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are avoided (Table 1) and very severe aggressive environments are excluded. On the other hand, when long term durability is needed - as for *RCS* with an expected service life of 100 years or more - one cannot ignore the mechanism of formation of microcracks in the concrete cover and their subsequent transformation into macrocracks due to the corrosion of the reinforcement. Moreover, even when ordinary-durability *RCS* are required, microcracking of concrete should be avoided in case of very severe environmental exposure (e.g., the tidal zone in maritime works or frequent freezing and thawing accompanied by de-icing salt treatments). The steel reinforcing bars, specifically used to counteract the low tensile strength of concrete and the possible failure of plain concrete structures subjected to tensile or flexural stresses in service, paradoxically have become the main reason of concern for the long-term durability of *RCS*. Concrete structures, even when microcracked by restrained thermal or drying shrinkage, could theoretically perform as long-term-durable structures, in the absence of reinforcing bars, provided that a low *w/c*, an adequate air-void system, effective compaction, and proper curing were adopted. However, today's concrete structures cannot be designed without the use of steel reinforcing

The next section examines some conceivable scenarios to produce *RCS* with long-term durability, as alternatives to those currently used for *RCS* with ordinary durability.

FUTURE SCENARIOS FOR LONG-TERM-DURABLE RCS

For new *RCS* to be competitive, in terms of durability, with ancient unreinforced structures, three main scenarios are possible:

- a) Improvement in the corrosion behavior of the metallic reinforcement,
- b) Use of non-metallic reinforcement,
- c) Higher tensile strength and/or more ductility of special concrete mixtures, and
- d) Surface coatings for concrete protection.

a) Metallic Reinforcement with improved corrosion behavior: The improvement should consist in a significant reduction of the corrosion process of the reinforcement, even when, due to exposure to aggressive environments, the reinforcement is **embedded in a micro-cracked cement matrix**. In other words, even in the absence of the passivating action of the cement-matrix, the reinforcement should resist **by itself** the corrosion promoted by CO_2 or Cl^- ions and fed by humid air (O_2 and H_2O). The corrosion rate - in terms of reduction in the cross section of the reinforcement - should be as low as few $\mu\text{m}/\text{year}$ in order to achieve two important objectives:

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- (i) Reliable safety, from a structural point of view, of *RCS* exposed to aggressive environments for a long service life
- (ii) Absence of any disruptive action of the reinforcement which could transform microcracks into macrocracks in the concrete cover and then be detrimental to the long-term durability of the *RCS*.

Based on the available experiential knowledge (13), the following options are theoretically available for the long-term protection of metallic reinforcing bars:

- Use of corrosion inhibitors as concrete admixtures,
- Protection of the reinforcement with a coating,
- Change in the composition of reinforcing bars to more durable alloys (e.g., stainless steel), and
- Cathodic protection methods.

However, so far none of the above protection methods appear to be sufficient to provide corrosion protection unless a crack-free long-term durable concrete is used (Table 3). For instance, the use of corrosion inhibitors does not protect the reinforcement from corrosion when cracked *RCS* are exposed to sea water or any other Cl^- source (14). In spite of a low-porosity cement matrix, concrete microcracks and cracks (> 0.1 mm wide) act as preferential paths for the aggressive agents which, hence, have direct access to the surface of the reinforcement close to the crack tip.

Coated reinforcements, and in particular epoxy-coated reinforcements (*ECR*), can lead to an "extra life" in the functionality of *RCS* (15) by prolonging the service life defined as the time from the construction of the structure to the total loss of its functionality (Fig. 2). However, due to the diffusion of water molecules through the coating and the subsequent loss of its adhesion to the steel surface (16), an underfilm corrosion can occur in uncracked *RCS* and, to a greater extent, in microcracked structures.

The use of galvanized (17), copper-clad (18), stainless steel-clad (19) or solid stainless steel bars (20) can prolong the service life due to the shift in the Cl^- threshold above which corrosion of reinforcing bars is promoted. This threshold can change from 0.4% by mass of cement in ordinary steel to about 1.2% or 3.5% for galvanized or stainless steel-clad and solid stainless steel, respectively. However, in very severe exposure conditions (such as tidal zones, structures semi-immersed in a Cl^- rich ground, or structures treated with de-icing salts) where high chloride content can be accumulated, the threshold value can be reached in a relatively short time particularly in microcracked *RCS*. Therefore, galvanized or stainless steel appears to be successful for long-term durability of *RCS* provided that the constructions are exposed to carbonation only in the absence of chloride.

Cathodic protection of the reinforcement (21), by using an impressed electrical current, seems to be a very promising method even if, because of the high financial investment in its installation, so far it is more used in

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rehabilitation works rather than in new constructions*. Theoretically, the protection of *RCS* from corrosion is just a question of a sufficient decrease in the electrochemical potential of the reinforcement (cathode), whatever Cl^- penetration occurs through the cement matrix or the preferential paths of microcracks. However, the decrease in the cathodic potential raises both the maintenance cost during the service life and the risk of other concurrent electrochemical processes (e.g., cathodic reduction of 2H^+ to H_2 and consequent embrittlement of steel in prestressed concrete structures). Presently, the main concern for the cathodic protection method is in the proper distribution of the auxiliary anode which should be applied close to the cathodic steel reinforcement. With respect to the past experience in the traditional cathodic protection of steel in soil or in sea water, there are clear logistical problems which are specific to *RCS*. In particular, due to the concrete resistivity, the flow and distribution of cathodic current from the anode to the steel reinforcement network can be obstructed. Therefore, the reinforcement in close proximity of the anode receives more current and is over-protected, whereas more distant reinforcements receive only a small fraction of the impressed current and remain under-protected (22). In conclusion, it seems that cathodic protection can be a successful and reliable method to provide long-term durability for new constructions, provided that a specific and tailor-made electrochemical design is adopted concurrently with the structural design of the *RCS*.

b) Use of non-metallic reinforcement: Fiber Reinforced Polymer (FRP) bars in general consist of organic or inorganic high-strength fibers in a resin matrix. The most commonly used FRPs for civil engineering applications are carbon (CFRP), aramid (AFRP), and glass (GFRP). The use of FRP reinforcing bars to replace steel reinforcing bars is one of the many techniques to improve the corrosion resistance of *RCS*. They are utilized as reinforcement for reinforced and prestressed concrete members, as well as for repairing or strengthening existing concrete structures (23, 24).

Although the use of FRP reinforcing bars is presently impeded by the lack of design procedures for practicing engineers, the FRP bars are certainly on the horizon of new *RCS*, and ACI is expected to issue soon a design guide for these bars (24).

c) Special concretes with higher tensile strength and/or ductility: Long-term durability (crack-freedom) can be achieved in *RCS* by an increase in the tensile strength and/or a decrease in the modulus of elasticity of the concrete (25). Concrete cracks when the tensile stress induced by restrained thermal or drying shrinkage exceeds its tensile strength. Due to creep of concrete, some of the stress is relieved and it is the residual stress - after stress relaxation from creep - that determines whether cracking will occur (Fig. 3). In general, high compressive strength concretes are intrinsically prone to microcracking, even to a greater extent than ordinary strength concrete, since the increase in the

* According to Pedferri (21) "cathodic prevention" should be used for new *RCS*, whereas "cathodic protection" would be related to the rehabilitation of deteriorated constructions.

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compressive strength is accompanied by an increase in the elastic modulus which is higher than that in the tensile strength. Therefore, unless some specific ingredients are used to manufacture special concretes, pursuing a crack-free concrete with high tensile strength (Fig. 4) or low elastic modulus (Fig. 5) does not seem to be a practicable approach.

These special concretes exist in form of polymer-modified concretes characterized by a monolithic co-matrix in which the organic polymer matrix and the cement gel matrix are homogenized (26). In general, polymer modified concretes show a significant increase in tensile and flexural strengths and a negligible increase in compressive strength. This is due to the contribution of the polymer phase interpenetrating throughout the cement phase. Figure 6 illustrates the influence of the addition of styrene-butadiene rubber (*SBR*) latex on the flexural strength of concretes with and without steel fibers (26). It seems that polymer-modified concrete, particularly with steel fiber additions, is a promising material for preventing microcracks induced by restrained length changes and, therefore, for assuring long-term durability of *RCS*. However, it is seldom employed because it is very expensive compared with traditional concrete mixtures. The polymer modified concrete has been used in Japan (27), USA (28) and Europe (29) only in some special applications, such as bridge deck overlays or patching work.

d) Surface coatings for concrete protection: Because of the high cost of polymer modified concrete, polymer-modified mortars have been developed to act as a protective coatings as thin as 1-2 mm on the surface of the concrete substrate. Due to the relatively low thickness of these coatings, the cost increase related to the coating application is much lower compared with using polymer-modified concrete in bulk for *RCS*. On the other hand, the cost increase of the coating application becomes negligible for *RCS* with long-term durability if one considers the reduction in the cost for rehabilitation of these structures, particularly when subjected to very severe environmental exposures.

There are two main types of surface coatings depending on their modulus of elasticity: the rigid coatings, particularly epoxy- and urethane-based material (30), are designed, for instance, to withstand wheeled traffic, but they must be replaced on a scheduled basis. Without traffic, these coatings would be expected to last longer; however, due to their intrinsic rigidity these coatings when exposed to thermal changes, cannot deform to bridge the cracks in the concrete substrate and guarantee crack-freedom and long-term durability of *RCS*. From this point of view flexible coatings perform much better provided that they are not designed to withstand abrasion or impact stress.

The original idea pursued by Swamy et al. (31) was to employ a flexible coating combining an acrylic elastomer (based on an aqueous 2-ethylhexyl polyacrylate emulsion) with mineral filler and inorganic pigments. Subsequent developments by Coppola et al. (32) led to flexible mortars made from the same acrylic polymer aqueous emulsion combined with cement and fine aggregates. In these materials, thanks to the consumption of the water of the emulsion by

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reaction with the cement, the hardening time is reduced. These coatings are by themselves resistant to the aggressive agents present in the environment (water, CO_2 , Cl^- , and SO_4^{2-} ions), are sufficiently flexible so that they can deform and bridge the un-avoidable cracks in the rigid concrete substrate, and can retain these characteristics regardless of the environment in which they are placed.

Experimental results on the performance of these flexible coatings are available as a function of the exposure time in the laboratory, underwater, and exposed to outdoor natural environment (31, 32). It should be pointed out that, similar to field experience with reinforcements having improved corrosion behavior, also in this case the available data refer to exposure times of only a few years. In conclusion, no direct field experience of real long-term durability (> 50 years) is available at this time. However, in contrast to what happens to new types of reinforcement embedded into concrete, the long-term behavior of flexible surface coatings can be directly monitored and, in case of failure, the coating can be reapplied or modified.

CONCLUSIONS

Modern reinforced concrete structures are less durable than the ancient unreinforced concrete structures because of the corrosion risk of the steel reinforcement embedded in a brittle concrete.

Human negligence in adopting available experiential knowledge on proper design, placing, and curing of concrete is considered to be responsible for the lack of ordinary durability (up to 50 years of service life). However, the lack of human negligence is only a pre-requisite to achieve long-term durability (100 years or more).

Initial microcracks, produced by restrained thermal and hygral length changes or loadings in service, act as preferential paths for the environmental aggressive agents. This is detrimental to the long-term durability of reinforced concrete structures even when proper concrete mixtures are designed, placed, compacted, and cured.

The shift from ordinary to long-term durability can be achieved through one of the following developments:

- Improvement in the inherent corrosion resistance of the reinforcement.
- Increase in the tensile strength and strain capacity of the concrete.
- Use of surface coatings to protect the concrete substrate from penetration of the aggressive agents.

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TABLE 1—MAIN CAUSES OF DETERIORATION IN REINFORCED CONCRETE STRUCTURES (RCS) DUE TO HUMAN NEGLIGENCE.

HUMAN NEGLIGENCE:	
First Type	Second Type
• Inadequate materials: - cements - aggregates - chemical & mineral admixtures • Improper mix-proportioning: - w/c - air void system • Inadequate structural design: - cover thickness - position & density of rebars - form & size of elements • Careless execution techniques: - slump & slump loss - placement - compaction - curing	• ASR monitoring in field experience: - alkali-reactive aggregates - high alkali cement • DEF induced distress: - high sulfate clinker - microcracking - exposure to water

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TABLE 2—MAIN CAUSES OF DETERIORATION IN RCS DUE TO THEIR INHERENT VULNERABILITY.

INHERENT VULNERABILITY OF RCS:	
Concrete	Steel reinforcement
• poor tensile strength • high modulus of elasticity • prone to microcracking produced by thermal and drying shrinkage • microcracks as preferential paths for aggressive environmental agents (humid air, Cl^- and SO_4^{2-} ions)	• corrosion when exposed to humid air or Cl^- ions penetrating through microcracks • expansive-disruptive nature of the corrosion process causing macrocracks in the concrete cover

TABLE 3—IMPROVED PROTECTION OF METALLIC REINFORCEMENT AND RELATED LIMITS IN SUCCESSFUL APPLICATIONS.

Type of protection:	Limits in successful applications
• Corrosion Inhibitors	• They do not work in microcracked concrete
• Epoxy Coated Reinforcement	• Loss of adhesion due to water diffusion and under-film corrosion
• Galvanized Steel	• Corrosion occurs with Cl^- over 1.2% by cement mass.
• Stainless Steel-Clad • Solid Stainless Steel	• Corrosion occurs with Cl^- over 3-4% by cement mass.
• Cathodic Protection	• Tailor-made electrochemical design is needed for the specific distribution of the reinforcements

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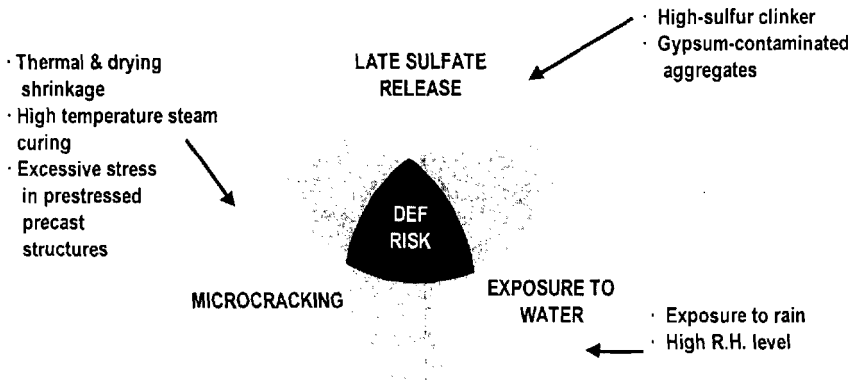


Fig. 1—Holistic model showing DEF damage as function of late sulfate release, microcracking, and exposure to water (11).

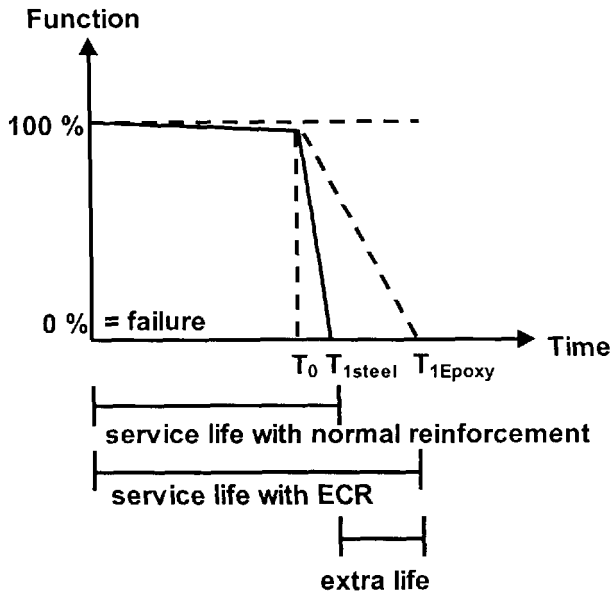


Fig. 2—Design concept for use of epoxy-coated reinforcement (ECR) as corrosion protection system (15).

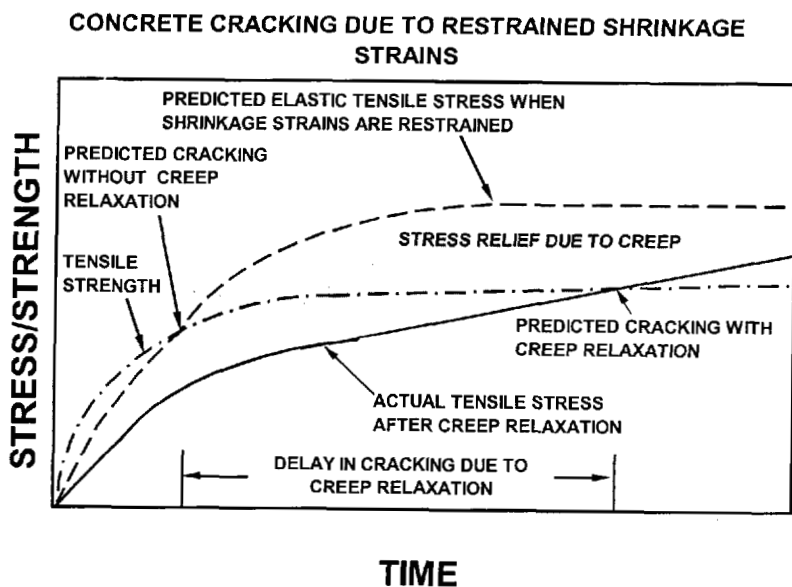


Fig. 3—Cracking of concrete due to lower tensile strength with respect to tensile stress (adapted from [25]).

CRACK-FREE CONCRETE RELATED TO HIGH TENSILE STRENGTH

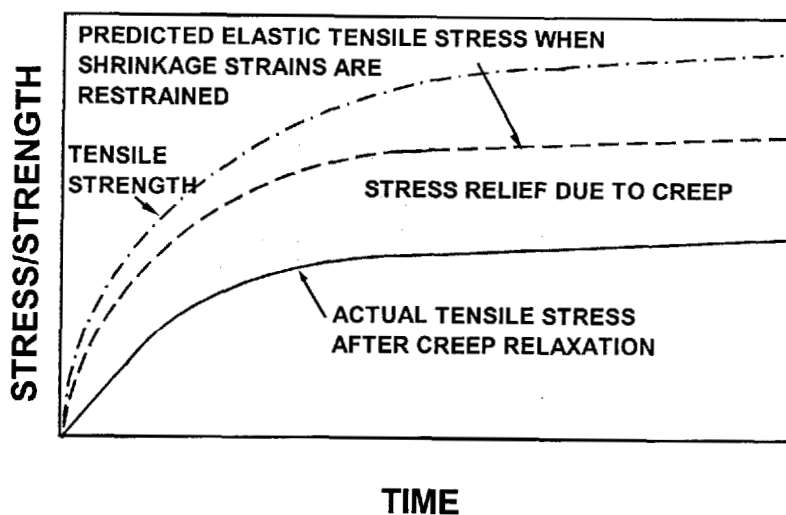


Fig. 4—Crack-free concrete due to tensile strength increase with respect to tensile stress.

CRACK-FREE CONCRETE RELATED TO LOW MODULUS OF ELASTICITY

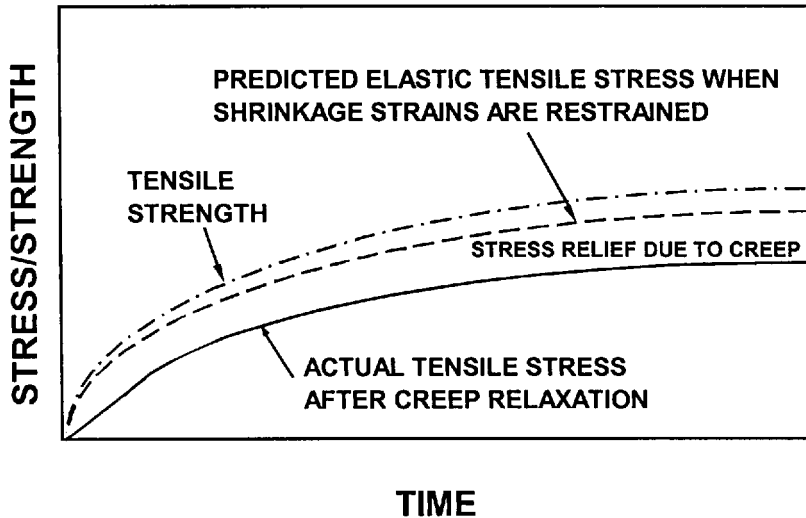


Fig. 5—Crack-free concrete due to tensile stress decrease with respect to tensile strength.

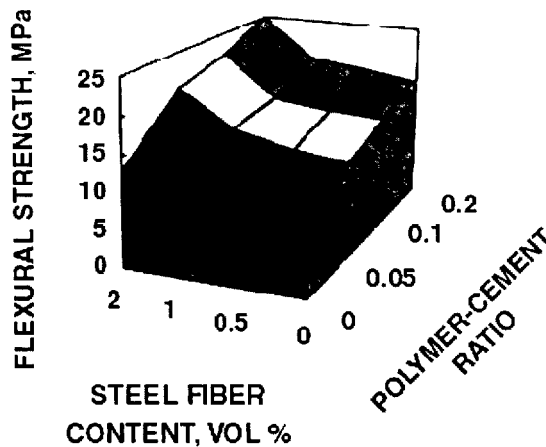


Fig. 6— Steel fiber content and polymer cement ratio versus flexural strength of latex modified (SBR) concretes (26).

SP 192-2

Monitoring Corrosion Risk in Concrete Structures—Review of 10 Years Experience and New Developments

by M. Raupach and P. Schießl

Synopsis: Since 1990 the Anode-Ladder-System has been used world-wide to monitor the corrosion risk of new concrete structures. This sensor-system is an embedded macrocell system indicating the depth of the critical chloride content initiating corrosion of the reinforcement. Subsequently the time-to-corrosion of the reinforcement can be determined continuously, enabling the owners of buildings to initiate preventive protection measures before damage like cracks and spalling occur.

However, this system can only be installed before placement of the concrete. Therefore a new sensor system has been developed to monitor the corrosion risk for the reinforcement also in existing or repaired structures. This new Expansion-Ring-System consists of 6 anode rings separated by sealing rings and a cathode bar, which are installed in small holes drilled into the concrete and connected by a special expansion mechanism. Actually it is in the stage of final testing in laboratories and pilot projects.

Keywords: cathodic protection; concretes; corrosion; reinforced concretes; steels

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INTRODUCTION

Corrosion of the reinforcement has become a major problem world-wide, for structures exposed to aggressive environments such as bridges, parking decks, tunnels, offshore structures or other buildings exposed to seawater or de-icing salts.

Many new systems and materials have been developed to repair the damage and to increase durability, e.g. high performance concretes. However, there is only limited long-term experience with these developments. Therefore new methods are needed enabling the owners of the structures to monitor the corrosion risk of the reinforcement making it possible to take protective measures *before* the damaging processes start. As is known, normally the costs for repair measures increase more than proportionately with time, when the reinforcement already has started to corrode.

The existing monitoring systems like reference electrodes only show the onset of corrosion and not the time-to-corrosion. Furthermore long-term-stability is not known for several types of reference electrodes. However, the sensor systems described in this paper are designed for long-term-measurements in the range up to 100 years to monitor the time-to-corrosion.

THEORETICAL BACKGROUND

The development of the corrosion monitoring sensors was based on an extensive research program on the main factors influencing chloride induced macrocell corrosion of steel in concrete (see e.g. (1)). More than 500 concrete specimens with different concrete compositions have been investigated under varying environmental conditions. These investigations have been carried out using macrocell current measurements between anodically and cathodically acting steel surface areas.

The operation of a macrocell consisting of a piece of black steel (anode) and a noble metal (cathode) is shown in Fig. 1. In chloride free and non-carbonated concrete, both electrodes are protected against corrosion due to the alkalinity of the pore solution of the concrete (passive state). The electrical current between both electrodes is negligibly low under such conditions.

If, however, a critical chloride content is reached, or if the pH-value of the concrete decreases due to carbonation, the steel surface of the so-called anode is no longer protected against corrosion. Provided that the selected cathode material is corrosion resistant in chloride contaminated or carbonated concrete and sufficient moisture and oxygen are available, oxygen reduction takes place at the surface of the so-called cathode-bar. The local separation of anodically and cathodically acting areas leads to an electron flow between the black steel

and the noble metal, which can easily be measured using an external cable connection.

Fig. 2 shows as the result of a demonstration test in the laboratory the electrical macrocell-currents flowing between a black steel anode and a stainless steel cathode embedded in two concretes with different w/c. The macrocells were embedded with a concrete cover of only 5 mm (0.20 in.) to initiate corrosion by applying a chloride solution on the concrete surface within a short period of time. The results of the macrocell current measurements showed that the critical chloride content reached a depth of 5 mm (0.20 in.) at the specimen with w/c = 0.7 about 80 days after concrete placement. This caused a significant increase of the macrocell current while the specimen with a lower w/c and a higher resistance against chloride diffusion remained passive.

BASIC PRINCIPLE TO DETERMINE THE TIME-TO-CORROSION

The basic measuring principle is to place electrodes into different depths related to the concrete surface and to measure the onset of corrosion of these electrodes one by one (see Fig. 3).

Using adequate calculation models, the time to corrosion can be determined at any time provided that the cover thickness of the reinforcement is known. However, the exact concrete cover can be verified using cover-meters.

DESCRIPTION OF THE ANODE-LADDER-SYSTEM

The standard sensor consisting of 6 single anodes is shown in Fig. 4 (see also (2)). Each of the 6 black steel anodes is positioned 50 mm (2 in) from the next one to prevent interactions between these measuring electrodes.

The cables are lead through stainless steel fixtures to the measuring device. The cable shaft within the stainless steel frame is filled with epoxy resin that acts as mechanical protection for the wires and an integrated PT 1000 temperature sensor. Two cables are connected with each single anode to get a redundant system. This allows the cables and cable connections to be checked after installation by short circuit tests.

The fixture is separated from the anodes by insulation. This geometrical design of the sensor ensures that the concrete cover above every single anode is not affected by parts of the sensor, and that the penetration of chlorides into the concrete and the carbonation process are consequently also not influenced.

The layout of the sensor system allows, besides the readings of the electrical currents as described above, also other measurements improving the information on the overall corrosion risk within the monitored structure:

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- The measurement of the potential between the anodes and the noble cathode gives further information on the corrosion behaviour of the reinforcement, especially including the availability of oxygen.
- The simultaneous measurement of the temperature by means of the incorporated temperature sensor allows a more detailed interpretation of the readings.
- The anodes are additionally used as measuring electrodes for AC resistance measurements at the different distances from the concrete cover. These readings are especially interesting e.g. to monitor the efficiency of coatings preventing water ingress into the concrete.

MEASUREMENTS AND EVALUATION

For the measurements mentioned above an automatic portable instrument has been developed. Using this battery-powered instrument the electrical currents, potentials, electrolytic resistances and the temperature are measured automatically after starting the measurement mode. The critical values are indicated simultaneously within the large display. The data of the measurements of 1.000 sensors can be stored and transferred to a personal-computer by the integrated serial port. For further evaluation of the data software is available, which builds up a database and shows the data versus time including alarm-values. This instrument can be seen in Figs. 12 and 13.

Fig. 5 shows, as an example, how the critical depths can be determined from the results of the measurements of an Anode-Ladder. It shows the standard plot produced by the software developed for the monitoring systems. To get a direct overview of all data, the time-dependent values of voltage of all anodes against cathode, el. currents 5 seconds after coupling of the anodes against the cathode, the AC-resistance between neighboured anodes and temperature of the integrated PT 1000 sensor are plotted on 1 page.

Looking to the electrical currents and voltages it can be seen, that the outer two anodes of the Anode-Ladder no. 1 and 2 indicate activity, while the other four deeper anodes indicate passivity. As the single anodes have been installed in 1 cm-steps, consequently the critical depth related to reinforcement corrosion is between 2 and 3 cm. To achieve this situation a chloride solution containing 1 % Cl⁻ was applied to the concrete surface beginning from 1 week after placement of the concrete over a period of four months using a concrete with w/c = 0.6 and 300 kg/m³ normal portland cement.

The limit values for activity of the single anodes under usual conditions are known from experience to be 150 mV for the voltage and 15 µA after a short circuit time of 5 seconds for the electrical current. For special conditions regarding concrete composition or environment, e.g. for permanently submerged concrete, other limit values are relevant, which can be determined

easily by calibration tests using the same concrete under the same conditions or evaluation of the time plots as shown in Fig. 5.

The following section describes the results from calibration tests for the use of the Anode-Ladder-System in high performance concrete with silica fume, fly ash and an extremely low water/cement-ratio.

CALIBRATION TESTS OF THE ANODE-LADDER-SYSTEM

Fig. 6. shows the result from the calibration tests of single reinforcement bars with the same geometry as of the single anodes of the Anode-Ladders in high performance concrete with addition of different chloride contents mixed to the fresh concrete. As reference concrete a poor-quality concrete with w/c 0.7 has been investigated with a chloride content of 1.5 % / cement.

To investigate additionally the most suitable short circuit time between the single anodes and the cathode bar, five different short circuit times between 1 sec. and 30 sec. have been tested.

As to be expected, it was found that the high performance concrete without addition of chlorides showed very low macrocell currents lower than 15 μA independently from the short circuit time. On the contrary, when 1 to 5 % chlorides/cement had been added to the concrete, considerable macrocell currents in the range of 50 μA have been measured. The effect that the el. currents didn't increase with chloride content can be explained by the limited size of the cathode bar, because the cathodic process seems to be the rate controlling factor for the investigated corrosion conditions (see e.g. /3/).

However, as it is not intended to measure absolute corrosion rates, but the critical depth of corrosion, the size of the cathode bar is sufficiently large to separate between the passive and active state of the so-called anodes. The sensors within the reference concrete show much higher currents in the range of more than 200 μA as result of the poor concrete quality.

Regarding the most suitable short circuit time Fig. 6 shows that generally all selected times could be used to be able to differentiate between passive and active state. As for practical reasons a short measuring time is most convenient, a short circuit time of 5 seconds has been defined as standard short circuit time.

APPLICATIONS OF THE ANODE-LADDER-SYSTEM

Since 1990 the Anode-Ladder-System has been used in different types of structures to monitor the corrosion risk for new reinforced concrete structures in aggressive environment world-wide. About 1000 Anode-Ladders have been installed into tunnels, bridges, foundations and other structures exposed to aggressive environment in more than 10 countries all over the world.

Typical installations are shown and briefly described in Figs. 7-10.

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Actual information of the sensor systems and new developments are published in the internet-website: www.sensortec.de.

DEVELOPMENT OF THE EXPANSION-RING-SYSTEM

As further development since 1997 the patented Expansion-Ring-System is intended to monitor the corrosion risk also for existing or repaired structures by inserting the sensors into drilled holes. The Expansion-Ring-System consists of the Expansion-Ring-Anode and a cathode-bar. Both electrodes are inserted into holes, which have to be drilled into the concrete surface.

The main measuring sensor is the Expansion-Ring-Anode (see Fig. 11). Similar to the six bars of the Anode-Ladder-System the Expansion-Ring-Anode consists of six measuring rings in different distances from the concrete surface, in 1 cm-steps from 1 to 6 cm. If the ingress of chlorides shall be monitored in smaller steps than 10 mm, two or more Expansion-Ring-Anodes can be installed at different levels, e.g. one sensor from 1 to 6 cm and one from 1.5 to 6.5 cm resulting in 0.5 cm steps.

Above, between and below these metal rings altogether seven insulation rings are established to seal the area between the rings, i.e. to prevent water or chlorides penetrating along the drilled surface deeper into the hole of the Expansion-Ring-Anode.

The cathode (see also Fig. 11) consists of two pieces of a platinum-coated bar to be expanded by a special mechanism to ensure a durable contact to the concrete.

The Expansion-Ring-Anode and cathode bar are connected to the concrete by turning nuts at the upper part of the sensors, leading to an expansion of the rings in a way that both sensors can be tightened sufficiently strong. During installation of the sensors the torque of the sensors will be measured and controlled to ensure that the tension between sensor and concrete is sufficient for a durable contact. During installation additionally the electrolytic resistance between the rings can be measured to check the electrical contact between the rings and the concrete. The Expansion-Ring-Anode is designed in a way, that the movements of the rings are limited to an extent, that no cracking of the concrete has to be expected.

In the head of the Expansion-Ring-Anode a socket is integrated, which allows to insert a plug of the measuring instrument. Between the measurements a sealing cap is put onto the socket to protect it from corrosion. Cables are lead from the measuring rings to this integrated socket inside of the Expansion-Ring-Anode. After production of the electrode all inner open spaces are filled with resin and sealing material to ensure that no water or chlorides can penetrate to the inner part of the sensor.

Laboratory tests have shown that the formation of rust at the first anodes after depassivation has an additional sealing effect. Therefore it can be excluded, that the depth of the critical chloride front is influenced by the presence of the sensors, because chlorides can only enter into the concrete besides the Expansion-Ring-Anode through the existing concrete.

FIRST APPLICATIONS OF THE EXPANSION-RING-SYSTEM INCLUDING MONITORING OF THE DURABILITY OF REPAIR MEASURES

Fig. 12 shows the Expansion-Ring-System installed into a bridge cap during measurement. Within this pilot project the sensors are tested under real conditions including frost action etc.

The Expansion-Ring-System can especially be used to monitor the durability of repair measures. Fig. 13 shows e.g. the installation of Expansion-Ring sensors into the columns of a parking structure, where a coating had been applied to prevent further ingress of chlorides and to dry the concrete increasing the electrolytic resistance of the concrete. The actual results show, that the concrete is drying out and no corrosion has to be expected. However, this situation is likely to change after several years.

FIELDS OF APPLICATION

The corrosion monitoring system has the following advantages compared to other monitoring methods:

- It is not only be seen, whether the reinforcement corrodes or not, but it can also be estimated, when the reinforcement will start to corrode.
- The corrosion monitoring system shows directly the depth of the critical chloride content and not the absolute chloride contents. Compared to the measurement of chloride profiles this is a decisive advantage, because as known from experience the interpretation of absolute chloride contents is often difficult. Therefore the installation of corrosion monitoring sensors can not be replaced only by taking concrete samples for chloride profiles.
- The use of the corrosion monitoring system is especially economic in not accessible locations, e. g. at the outer surfaces of tunnels or in the tidal zones of pier shafts.

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OUTLOOK

Laboratory tests and installations of about 1000 Anode-Ladders in the last 10 years on site have shown, that it is possible to monitor the critical depth related to reinforcement corrosion using the Anode-Ladder-System.

Since September 1998 the Brite Euram Project "Smart Structures" is running to develop an integrated monitoring system for concrete structures. One task of this project is the further development of the Expansion-Ring-System, especially optimisation of the installation procedure and testing in different concretes in laboratory and in critical locations of bridges.

The further results will be presented within the next years.

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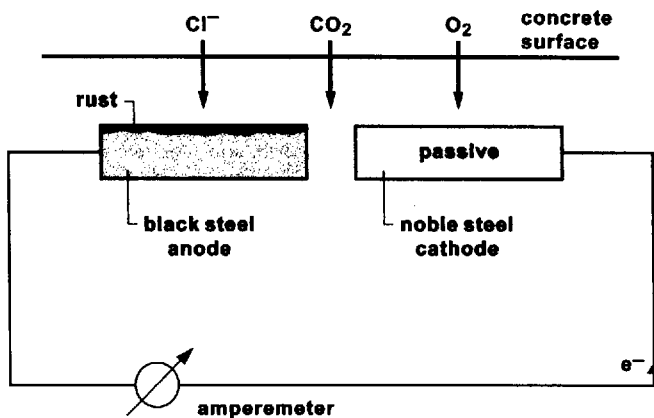


Fig. 1—Macrocell consisting of black steel anode and noble metal cathode.

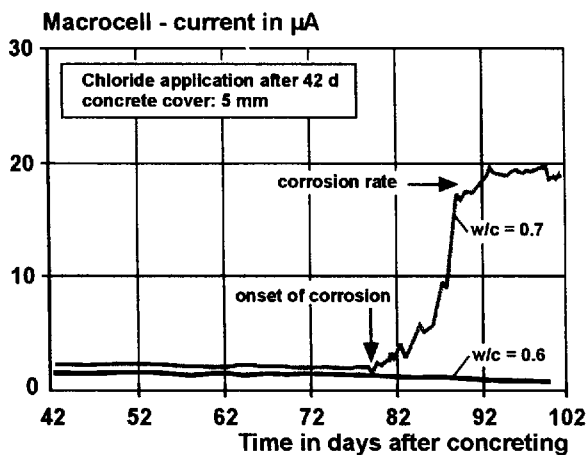


Fig. 2—Time-dependent behavior of galvanic currents.

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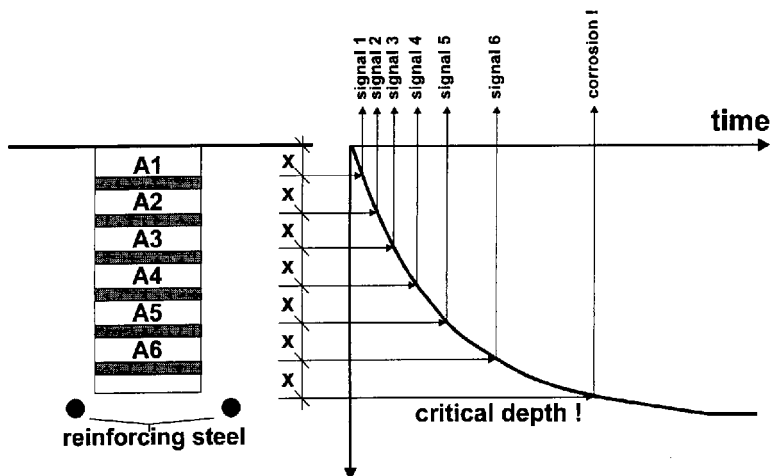


Fig. 3—Basic principle to determine time-to-corrosion.

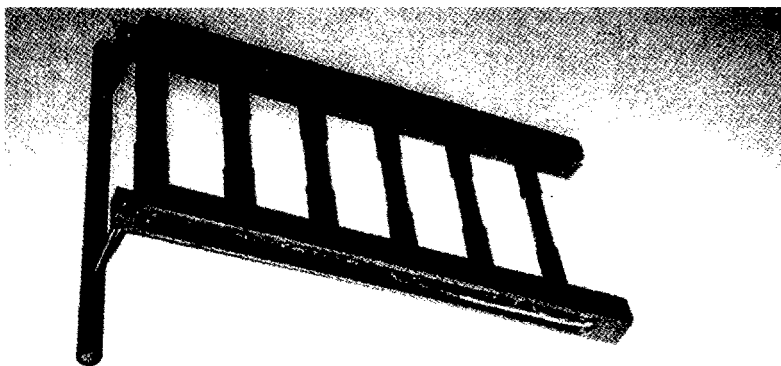


Fig. 4—Design of set of six anodes.

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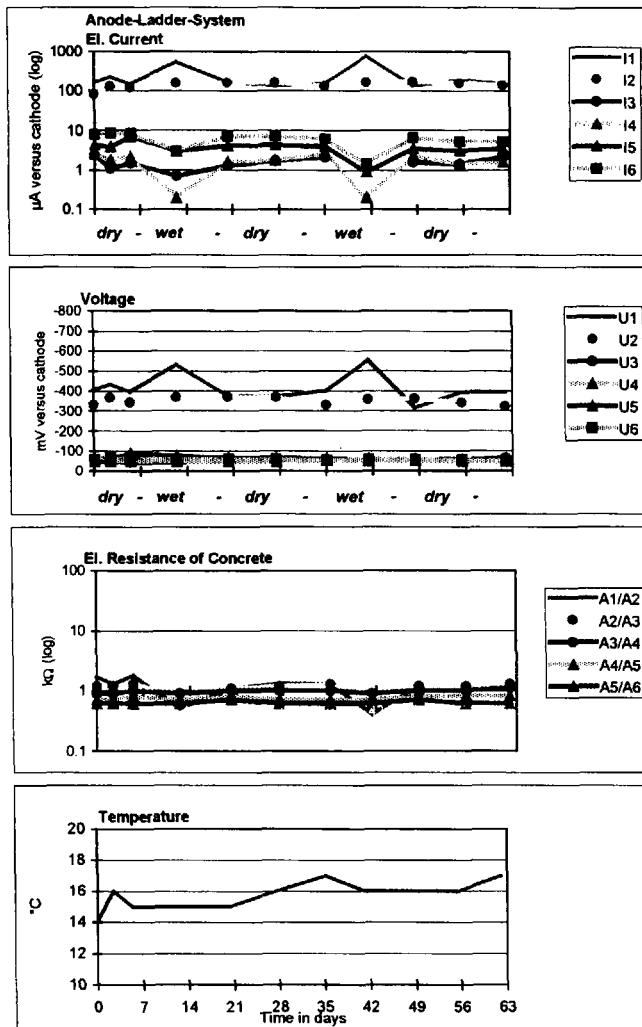


Fig. 5—Example of test results from laboratory of Anode-Ladder-System.

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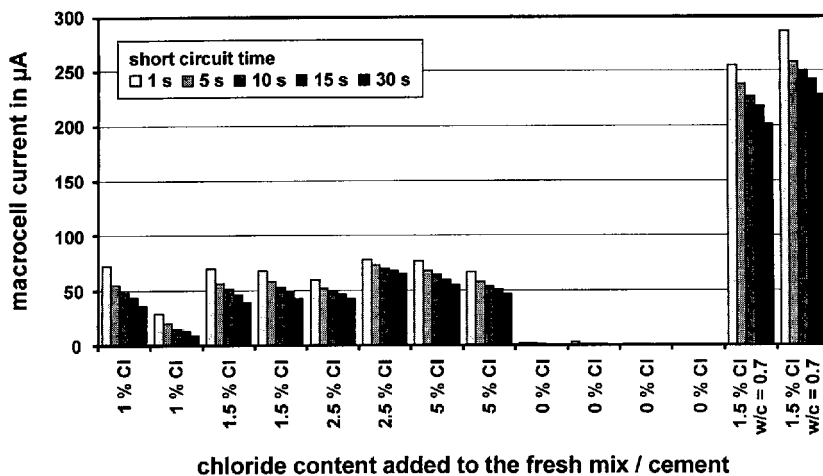


Fig. 6—Results from calibration tests of Anode-Ladder-System in high-performance concrete and concrete with poor quality.



Fig. 7—Two Anode-Ladders during installation into bridge-cap in Germany 1990, to monitor corrosion risk caused by deicing salts.

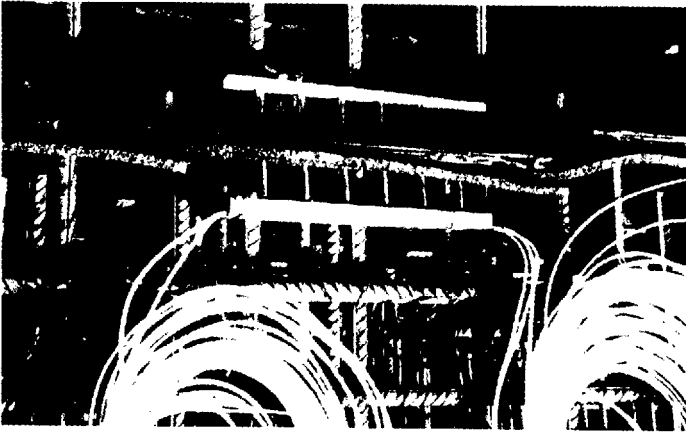


Fig. 8—Photograph of special Joint-Anode-Ladder installed in joint area between caisson and pier shaft.



Fig. 9—Photograph of Anode-Ladder within diaphragm wall welded at different levels to reinforcement below later water level, to monitor time-to-depassivation, and especially availability of oxygen.

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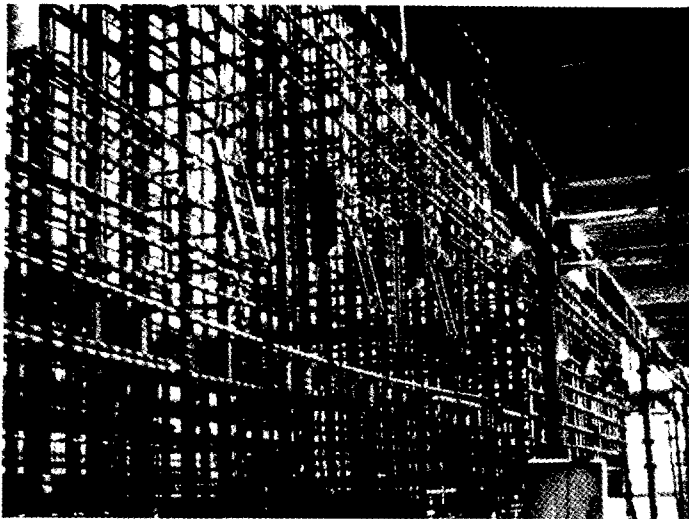


Fig. 10—Photograph of three Anode-Ladders within pylon installed in each monitoring location to receive information about scatter of ingress of critical chloride front into concrete initiating corrosion.

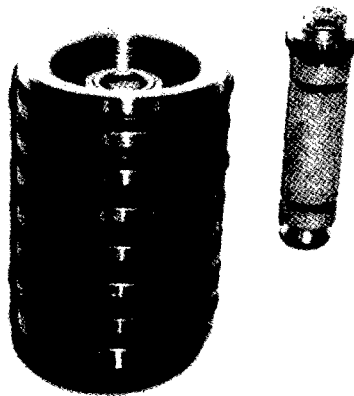


Fig. 11—Model of Expansion-Ring-Anode and cathode bar.

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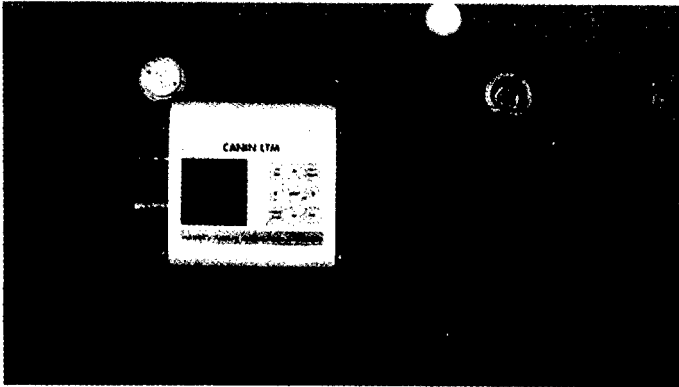


Fig. 12—Photograph of Expansion-Ring-System installed into bridge-cap, Germany, 1998, during measurement with removed protection cap.

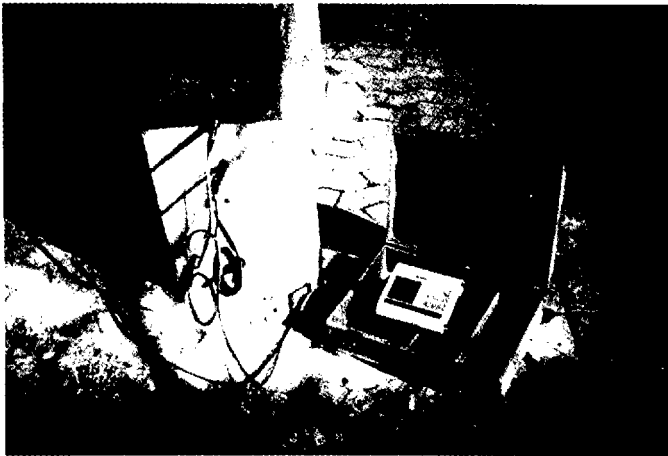


Fig. 13—Photograph of prototype of Expansion-Ring-System installed into column of parking garage to monitor durability of coating and corrosion behavior in area above coating.

SP 192-3

Curing Time Estimator for Portland and Portland/Fly Ash Concretes

By L. J. Parrott

Synopsis: The development of a curing time estimator is described: it is based upon an existing model of the microstructure and porosity gradients in the cover concrete that correlates well with relevant hydration and pore structure measurements. The same model also yields capillary porosities that correlate with measurements of compressive strength and water absorption rate. The objective is to provide a single method to estimate curing times for CEM I and CEM II (portland and portland/fly ash) concretes in a wide range of climatic conditions and achieve a consistent, well-defined measure of cover concrete performance. New hydration and pore structure measurements are briefly reviewed in relation to the existing model of microstructure and porosity in cover concrete. Recent developments regarding European standards for curing and concrete durability are considered. A criterion of capillary porosity in the matrix of cover concrete is used to unify the durability-related effects of curing period, cement type, water/binder and climatic conditions. The initial input to the estimator is the cement type to be employed. The next input is a maximum water/binder, as necessary to ensure durability under the expected exposure conditions; this automatically sets a target capillary porosity in the cover concrete, based upon recent curing period recommendations from European standards committees. Subsequent inputs define the climatic conditions in terms of exposure relative humidity and temperature history. The estimator calculates and tabulates capillary porosities in the cover concrete for a wide range of curing periods so that a period can be chosen without exceeding the target porosity. Capillary porosities for reduced water/binders, 95 and 90% of the input value, are also tabulated to illustrate the reductions in curing period that are possible with these higher concrete qualities. Examples are given to illustrate the effect of each of the eight inputs; water/binder, exposure relative humidity and cement type are the most influential. It is evident that in many cases control of cover concrete performance via curing options is limited relative to the control offered via small changes in the concrete mix proportions or alternative cements.

Keywords: cover; curing; exposure; fly ash; porosity

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INTRODUCTION

The aim of this paper is to develop a method for estimating curing requirements of concretes made with prEN 197 CEM I and CEM II cements [1] that are subsequently exposed to specified European climatic conditions. The single, transparent software method will enable designers and contractors to estimate curing times for a wide range of European cements, concretes and climatic conditions and achieve a consistent, well-defined measure of cover concrete performance.

The European standard for the execution of concrete structures and the chapter dealing with curing are currently under development. Recent CEN documents that cover the estimation of the periods of curing, needed for ensuring adequate serviceability of cover concrete, have been greatly simplified. They now account only for the effects of temperature, during the initial period when moisture loss is controlled, and of the rate of concrete strength development, such that curing is continued until the surface strength has achieved 40% of the compressive strength. Experimental and theoretical data indicate that the long-term performance of cover concrete depends upon continued cement reactions and microstructural development during the post-curing period when the concrete is exposed to natural climatic conditions. Continued reaction of the cement occurs mainly in the initial weeks of exposure, when the cover concrete is still partly moist, and depends upon the exposure humidity, exposure temperature and the type and reactivity of the cement used. The curing estimator is intended to provide a user-friendly method that takes account of these longer-term factors and is applicable to a range of European climates and materials. Use of the method will more consistently achieve the CEN Execution Standard objective of "ensuring adequate surface zone durability". The estimator also indicates alternative concrete qualities that provide the required cover concrete performance with reduced curing times. The proposed method is based upon modeling of cement hydration, porosity, moisture and properties in cover concrete. Measurements of key parameters under appropriate exposure conditions are used to validate the model.

The British Cement Association (BCA) has conducted experiments relating to the performance of cover concrete made with a wide range of European cements, concrete mix proportions and climatic exposure conditions: the experiments addressed permeability, diffusion, absorption, carbonation, chloride penetration and reinforcement corrosion [e.g. 2-6]. Parallel measurements of relative humidity, cement hydration and porosity were obtained and these data have been used to extend an existing portland cement model of

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cover concrete moisture and microstructure [7] and create an engineering software tool for estimating curing times. The measurements indicated that short-term atmospheric fluctuations of relative humidity had little effect at depths greater than a few millimeters from the exposed surface and supported the notion that modelling could reasonably be based upon an annual average atmospheric relative humidity. The capillary porosity was measured on partly dried samples by water adsorption from 91% relative humidity to saturation. The measured capillary porosity decreased with increasing cement hydration whereas the remaining gel porosity increased with cement hydration.

The model of cover concrete microstructure calculates, at a given depth from the exposed surface, the amounts of cement compounds reacted and the resultant volumes of reaction products. The cement compound reaction rates are dependent upon local relative humidity and temperature. The total porosity is calculated from the initial water filled volume and the increase in solid volume due to the products of cement reaction. The calcium silicate hydrate gel components are assumed to have gel porosities that depend upon the source compound, availability of pore space, moisture history and temperature history. The capillary porosity is the difference between total porosity and gel porosity. Four reports [8-11] indicate that the model successfully predicts measured values of relative humidity, cement hydration and porosity components at different depths from an exposed surface in CEM I (100% portland) and CEM II (70% portland and 30% fly ash) concretes made with a wide range of water/binders (0.35, 0.47, 0.59, 0.71 and 0.83), exposed to widely different climatic conditions (office, laboratory, outside sheltered, outside vertical and outside horizontal surface) and cured for 1, 3 or 28 days.

COMPARISONS BETWEEN STRENGTH MEASUREMENTS AND MODEL OUTPUT

The concretes for strength measurements were made with water/binders in the range 0.35 to 0.83 and were damp-cured for 1 to 545 days. The microstructural model was used to calculate the corresponding capillary porosities with temperature and relative humidity inputs set at 20°C and 98%, respectively. The measured strengths were plotted against the calculated values of capillary porosity to produce Figs. 1 and 2 for CEM I and CEM II cement concretes, respectively. The parabolic equations shown in these figures were employed for subsequent compressive strength modeling. The model has input factors for the reactivity of the fly ash and for each cement compound. Setting the reactivity factors at 0.2, 0.3, 0.5, 0.7, 1.2 and 2.5, for a CEM I mortar made with a water/cement of 0.5, gave a range of strength development curves corresponding to the prEN 197 strength classes 32.5 to 52.5R as illustrated in Fig. 3. Reactivity factors appropriate to each cement option were incorporated in the curing time estimator.

CEN EXECUTION STANDARD AND CURING CRITERIA

The current CEN position is to simplify the approach to curing such that a) the effects of post-curing climatic exposure upon cover concrete performance are ignored and b) it is independent of prEN 206 [13] and durability requirements. These simplifications appear to result from the lack of a comprehensive method able to reconcile the different European viewpoints that derive from engineering experience with mainly national cements, concretes and climates. The proposed approach is to utilize recent research that relates durability to the porosity and transport properties of cover concrete. The porosity results, [10,11] show the merit of using capillary porosity as a proxy for transport properties. Furthermore the correlation between porosity, transport properties and carbonation, for example are well known [2-6,14-25]: indeed a recent CEN inter-laboratory investigation of carbonation testing indicated a single, close relationship between carbonation depth and water absorption rate for a wide range of European cements [26].

The minimum curing times in a recent draft CEN document (June 1997) reflect the strength-based criteria developed during the preparation of prEN 206. The minimum curing periods for cover concrete temperatures of 15 to 25°C are 1, 2, 3 and 5 days for rapid, medium, slow and very slow rates of concrete strength development. The range of curing periods is consistent with that for UK practice and in British Standard BS8110, 1997 [25]. The concrete strength development rates are intended to correspond to 2 day: 28 day strength of greater than 0.5, 0.3 to 0.5, 0.15 to 0.3 and less than 0.15. Unfortunately these four strength development rates do not correspond directly to the six cement strength classes listed in prEN 197 [1] and employed for Fig. 3.

Table 1 shows the 2 day: 28day strength ratios from Fig. 3, the corresponding strength development classes according to the June 1997 CEN draft curing document and the corresponding estimated curing periods for a concrete temperature of 20°C. The subsequent columns show, for 0.4, 0.5 and 0.6 water/cement concretes, the calculated capillary porosities at the end of curing and after a further exposure for one year in a typical northern European climate (85% relative humidity) at a depth from the exposed surface of 10mm. The depth of 10mm was selected to give a porosity close to the average over a typical cover depth of 30mm. There is little consistency between cement strength classes, for any given water/cement, in the porosities at the end of curing, with a bias in favour of slow-hardening cements: they would give a more porous cover concrete. The results for one year of exposure lead to a similar conclusion. The porosity reduction after the end of curing, typically 0.16 to 0.26 was very significant in relation to the capillary porosity at the end of curing, from 0.16 to 0.50. This emphasizes the need to account for the microstructural development that takes place in the post-curing period and its dependence upon the cement reactivity, exposure conditions and water/cement. The average 1 year capillary porosities for 0.4, 0.5 and 0.6 water/cements from Table 1 were 0.013, 0.102 and 0.197, respectively. Similar calculations for other water/cements listed in prEN 206 [13], 0.45, 0.55 and 0.65, yielded average porosities of 0.054, 0.150 and 0.241,

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respectively. Exposure relative humidities lower than the 85% assumed for Table 1 would not change the capillary porosities at the end of curing but the values after 1 year would be greater and the performance of the cover concrete would be inferior. Thus the average one year porosities are regarded as conservative performance targets. Use of this longer-term porosity enables the benefits of damp or warm climatic conditions, low water/binders and reactive cements to be accounted for with a single, consistent criterion of cover concrete quality.

CURING ESTIMATOR

Target capillary porosity and output format

The maximum water/binder (w/b) required in prEN 206 to ensure durability is used as an input to the estimator and yields a capillary porosity target (cpt) in line with that given in the previous section. This is achieved using a simple, linear relationship:

$$\text{cpt} = 0.961 (w/b - 0.395)$$

After further inputs relating to climatic conditions and cement reactivity the estimator generates values of capillary porosity in the outer 30mm of cover concrete, for curing times in the range 1 to 56 days and for an input water/binder. The input water/binder need not be the maximum prEN 206 value that generates the target porosity. The estimator also generates capillary porosities versus curing times for two reduced water/binders, 95% and 90% of the input water/binder: this illustrates the reduced curing times that are consequently possible without an increase in the capillary porosity of the cover concrete. An example of the output format is given in Table 2 for a water/binder of 0.6. This example of output information shows that, for the given inputs, unrealistically prolonged curing would be required to achieve the target porosity with a water/binder of 0.6. However, even a small reduction of water/binder to 0.57 would enable the target porosity to be achieved with very short curing times. The other seven inputs and an indication of their effects are discussed in the following sections. The discussion is based upon variations of one input at a time and other inputs set at the values used in Table 2, apart from the exposure relative humidity; this is set at 80% unless otherwise specified. The capillary porosity values are reported for an age (curing time plus exposure time) of one year.

Exposure relative humidity

The influence of relative humidity is illustrated in Fig. 4; the capillary porosity in cover concrete diminishes with increasing relative humidity due to the dependence of binder reactions upon moisture availability. Hydration and pore structure development are very limited if the relative humidity drops below 80% [e.g. 27,28] but drying, even at depths of only 10mm, is buffered by the overlying concrete [3,6]. Thus there is not a stepwise variation in capillary porosity at an exposure relative humidity of 80%. However, as would be expected, there is a clear benefit from prolonged initial curing at lower exposure relative humidities. Concrete is seen to be almost self-curing at exposure relative humidities above 85%.

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Time of casting and temperature effects

A change in the casting day is associated with sinusoidal, seasonal changes of temperature with the highest temperatures occurring at 182 days (i.e. mid-year). The mean annual temperature can be varied but, for the present discussion, it is set at 12°C. The input for seasonal temperature half-range is set at 10°C so that the average daily temperature varies from a low of 2°C to a high of 22°C. Superimposed on the seasonal variation is a diurnal variation of temperature with a half-range of 5°C. Thus, with a casting day of 182 days, the temperature initially ranges from 17 to 27°C during a day and, 182 days later, the temperature ranges from -3 to 7°C during a day. Fig. 5 indicates that, for a given curing period, the effect of changing the casting day is very small, with only a slight disadvantage associated with casting at the coldest time of year. Any initial slowness in cement reaction is countered over the calculation period of a year by a higher subsequent rate of cement reaction, drying being slow enough at a depth of 10mm not to cause major impairment of hydration and pore structure development. Increases in the duration of curing beyond 3 days are necessary to give significant capillary porosity reductions; this limited effect is partly due to the exposure relative humidity being set at 80%. The effect of changing the casting hour was found to be minimal. Increasing the annual mean temperature from 6 to 15°C reduced the capillary porosity in the cover concrete by about 8%. Changes in the seasonal temperature range had an insignificant effect upon capillary porosity.

Water/binder, cement type and strength class

The effect of water/binder changes, in the range 0.4 to 0.7, upon capillary porosity in cover concrete is illustrated in Fig. 6. It is evident that the water/binder is a very influential input and a small reduction of about 0.025 produces reductions of porosity that are associated with an increase from 1 to 56 days in the curing period. With exposure relative humidities lower than 80% the significance of curing period could be two or three times greater.

The effects of cement type and cement strength class, in the range 32.5 to 52.5R, upon capillary porosity in cover concrete are illustrated in Figs. 7 and 8. The advantage of increasing the strength class upon capillary porosity is significant but less so than for changes of water/binder or exposure relative humidity. Again it is evident that the effect of curing period is relatively limited although, with CEM 2 cement, there are benefits of porosity reductions with prolonged curing to 56 days.

The effects of cement type and cement strength class upon capillary porosity in cover concrete are affected by the exposure relative humidity: Figs. 9 and 10 relate to an exposure relative humidity of 50% in contrast to the value of 80% used for Figs. 7 and 8. Capillary porosities are much higher for both cements due to faster drying of the cover concrete and associated low levels of cement reaction. Furthermore the beneficial influence of lengthening the curing period is now clearly visible, particularly in Fig. 10 for the concrete made with CEM 2

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cement. However, even with prolonged curing the porosities are substantially higher than those in concretes exposed at 80% relative humidity.

DISCUSSION

The previous discussion indicates that selection of a water/binder offers a greater degree of engineering control over the one year capillary porosity in cover concrete than does choice of cement type or strength class. Increasing the curing period, even to 56 days, leads to relatively modest porosity reductions with an exposure relative humidity of 80%. The benefit of curing is more evident with a lower exposure relative humidity of 50% and in concrete made with CEM 2 cement. Exposure relative humidity is the dominant climatic effect with regard to the one year capillary porosity in cover concrete: this is because cement reaction rates are virtually insignificant below 80% relative humidity. On the other hand reductions of temperature slow down the rate of cement reaction during the colder season but reactions do not cease; indeed they increase during the warmer parts of the year. Over the course of one year, at least with exposure at 80% relative humidity, much of the potential for cement reaction will then be realised.

This paper illustrates the effects of only a limited portion of the possible input combinations, as an aid to the understanding and use of the curing estimator. There are eight inputs that can each be set at numerous levels and there are seven curing periods at which capillary porosity values are generated. Thus the trends shown in this paper should not be regarded as generally representative. The estimator should be used to examine and compare alternative combinations of inputs relevant to each concrete engineering project. One valuable function is to indicate the trade-off between curing period and concrete mix design such that the quality of cover concrete is maintained when short curing times are required to ensure that projects are completed on time. It is evident that the target capillary porosity may not always be achieved with the prEN 206 maximum water/binder and with reasonable curing periods, perhaps due to dry climatic conditions or slowly reacting cements. In such cases the most practicable solution is to use a concrete made with a reduced water/binder: the resultant changes in concrete mix proportions will normally be small and of minimal materials cost.

USING THE CURING TIME ESTIMATOR

This section explains how the estimator software is used; it is organized on a screen by screen basis. The first screen requests a title and then expands to request the choice of cement type and strength class; currently there are ten options. The second screen requests a maximum water/binder that is consistent with durability requirements; those given in prEN 206 [13] are a useful guide. The input value generates a target capillary porosity in the binder phase of the cover concrete as explained earlier. An input water/binder is then requested after a high default option is provided. The input value will generally need to be smaller than the maximum value in order to generate capillary porosities no larger than the

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target porosity. Under dry exposure conditions or with slowly reacting cements the input water/binder will need to be significantly smaller than the maximum value. The third screen requests inputs for climatic data relating to the exposure relative humidity and temperature history expected during the first year after casting. These should be based upon local meteorological statistics. Default values are provided and often only the relative humidity and, perhaps, the annual mean temperature will require amendment. Completion of the inputs to the third screen initiates calculations and produces to the fourth screen. The fourth screen offers three simple options: 1) re-run the program, 2) display the results for this run or 3) exit from the program. The display option leads to the final screen that has the format previously exemplified in Table 2. It may be necessary to rerun the program with new inputs to achieve a cover concrete capillary porosity that is less than the target value with acceptably short curing periods.

ACKNOWLEDGEMENTS

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