

CANMET

Durability of Concrete

**Proceedings
Seventh CANMET/ACI International Conference
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V. M. Malhotra
Editor



American Concrete Institute®
Advancing concrete knowledge

Seventh CANMET/ACI International Conference on Durability of Concrete

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PREFACE

The Canada Centre for Mineral and Energy Technology (CANMET) of Natural Resources, Ottawa, Canada, has played a significant role in Canada for over 40 years in the area of durability of concrete. CANMET, in association with the American Concrete Institute and the Institute for Research in Construction/National Research Council, Ottawa, sponsored the First CANMET/ACI International Conference on Concrete Durability held in Atlanta, Georgia, April 27-May 1, 1987. The refereed proceedings of the Atlanta conference and Montreal conference (the Second CANMET/ACI International Conference on Concrete Durability, held August 4-9, 1991) were published as ACI SP-100 and ACI SP-126, respectively. Unlike the first conference, this second conference was not named after any individual(s), and the future conferences in this series would follow this precedent.

In 1994, CANMET, in association with the American Concrete Institute and several other organizations in Canada and France, sponsored the Third International Conference on the subject. The conference was held May 22-28, 1994, in Nice, France. The conference proceedings were published as ACI SP-145.

In 1997, CANMET, in association with the American Concrete Institute and several other organizations in Australia, sponsored the Fourth International Conference on the subject. The conference was held August 17-22, 1997, in Sydney, Australia. The two-volume proceedings of the conference, consisting of 81 refereed papers, were published as ACI SP-170.

In 2000, CANMET, in association with the American Concrete Institute, the Japan Concrete Institute, and several other organizations in Spain and Canada, sponsored the Fifth International Conference on the subject. The conference was held June 4-9, 2000, in Barcelona, Spain. The two-volume proceedings of the conference, consisting of 73 refereed papers, were published as ACI SP-192.

Concrete. The conference was held June 1-7, 2003, in Thessaloniki, Greece. The proceedings of the conference, consisting of 69 refereed papers, were published as ACI SP-212.

In 2006, CANMET, in association with several other organizations in Canada and the U.S., sponsored the Seventh CANMET/ACI International Conference on Durability of Concrete. The conference was held in Montreal, Canada, on May 28-June 3, 2006. More than 75 papers were peer reviewed in accordance with the policies of the American Concrete Institute. The proceedings of the conference, consisting of 50 refereed papers, were published by the American Concrete Institute as ACI SP-234.

In addition to the papers that have been published in the refereed proceedings, more than 50 other papers were presented at the conference. A number of these were published as supplementary papers in a special volume.

During the conference, special sessions were held on subjects with sulfate attack on concrete and high-performance lightweight concrete. Some of the papers related to these subjects were published in the supplementary volume.

Thanks are extended to more than 15 review panel members who met in Budapest, Hungary, in 2002 to review the papers. 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 their finalized manuscripts.

The assistance of A. Bilodeau, Chair of the audio-visual review panel, is gratefully acknowledged. Thanks are also extended to P. Gupta and C. Mansfield-Joiner for their help in processing the manuscripts. The contributions of the ACI staff for their help in publishing the proceedings on time is also recognized.

As an integral part of the conference, a special symposium was held to

on time is also recognized.

As an integral part of the conference, a special symposium was held to honor Professor K. Sakata of Japan for his outstanding contributions in the broad area of concrete design and technology over the past 20 years. The proceedings of the symposium were published as a separate volume.

V.M. Malhotra, P.Eng.

Editor

Chair, Seventh CANMET/ACI International Conference
on Durability of Concrete

May 2006

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Durability of Concrete – The Zigzag Course of Progress

by P.K. Mehta

Synopsis: Concrete design and construction practices today are essentially strength-driven. However, due to escalation in the repair and replacement costs of structures and a growing concern about sustainability of the concrete industry, more attention is being paid now to durability issues. An overview of the state of the built infrastructure in the world shows that the current methods for achieving considerable enhancement of durability of concrete structures are proving inadequate, especially with concrete members exposed to severe weather conditions. It seems that inhomogeneities in the microstructure of concrete are responsible for microcracks which, when subjected to tensile stress from weathering and loading effects during the service, grow into macrocracks. When the macrocracks, voids, and microcracks become interlinked there is a sudden increase in the rate of transport of water, carrying harmful ions and gases from the surface into the interior of concrete. This point marks the initiation and progressive deterioration of the material from one or more causes. A three-stage concrete damage process is discussed to show what changes in concrete technology are needed for radical enhancement in the durability of structures to be built in the future.

Keywords: concrete; damage; deterioration; durability; holistic; macrocracks; microcracks; microstructure; sustainability

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P. Kumar Mehta is Professor Emeritus in the Civil and Environmental Engineering Department at the University of California at Berkeley. Author or coauthor of numerous papers and four books in the field of concrete technology, he has received many awards, including ACI Wason Medal for Materials Research, and ACI Construction Practice Award. He held the Roy Carlson Distinguished Professorship in Civil Engineering at Berkeley, and upon his retirement, received the Berkeley Citation - the highest campus honor for exceptional contributions to his field and to the university.

INTRODUCTION

Approximately fifteen years ago, at the Second CANMET/ACI International Conference on Concrete Durability held in Montreal in 1991, the author presented a comprehensive paper on durability of concrete structures in North America¹. This paper was based on a review of several important publications on the subject during the previous 50-years period. The title of the paper was “Durability of Concrete - Fifty Years of Progress?” The title was derived from the conclusion that an unusually large number of concrete structures, many of them hardly a few years old, were found in a state of deterioration. This was attributed to developments since the 1940’s in the concrete industry that were driven by the speed of construction, and the strength of concrete rather than durability as the principal object of structural design and construction. Presented here is an update on the state of durability of concrete structures today. It seems that in spite of an accumulated knowledge base on how to build durable concrete structures, there has been essentially no progress on this issue.

There are compelling reasons why the concrete construction practice during the 21st century will have to be driven by durability rather than by the strength considerations. For instance, increasing carbon emissions, global warming, and a credible threat of climate change have brought to public attention the issue that sustainable economic development cannot occur without reducing the wasteful consumption of natural resources through radical enhancement of durability of manufactured products. Note the following excerpts from a March 2001 report, “Vision 2030: A Vision for the U.S. Concrete Industry”, issued by the Strategic Development Council, an organization of senior concrete industry executives dedicated to promoting the development of new technologies for which there is near-term commercial demand².

The industry will be able to offer better products, including durable constructed facilities with low maintenance cost....

Public concern will be responsibly addressed regarding climate change resulting from the increased concentration of global warming gases.

In this paper, I have used a holistic approach that involves a search for the root causes behind commonly known durability problems. This approach has led to identification of the three stages of concrete damage process. For radical enhancement of durability of concrete structures, it is proposed that the damage process must not be permitted to go

beyond the first stage, which can be achieved by understanding and control of the microstructure of concrete.

CONCRETE DURABILITY – A GLOBAL OVERVIEW

It was nearly 20 years ago when the U.S. National Materials Advisory Board, in 1987, sounded an alarm bell by reporting that concrete bridge-decks, built since the 1970's, were suffering from an epidemic of durability problems that would require billions of dollars to fix. Meanwhile, more cases of serious and premature deterioration of concrete infrastructure have been reported from around the world. To cite a few examples, Khanna et al.³ in 1988 reported the results of an investigation of premature deterioration in reinforced concrete piles of the Rodney Terminal at St. John in New Brunswick, Canada. The investigation revealed that thermal cracks during the pile manufacture were precursors to serious cracking from cycles of freezing and thawing, and from reinforcement corrosion. Gerwick⁴ in 1989 reported similar cases of premature cracking and deterioration in reinforced concrete lining of undersea tunnels in Dubai, Hong Kong, and Japan. In the 1990's, Shayan and Quick⁵ from Australia, and Collepari⁶ from Italy reported cases of premature cracking and deterioration in prestressed concrete railway sleepers.

In the United States, in response to widespread cracking of concrete bridge-decks, the construction practice moved towards the use of high-performance concrete (HPC) mixtures developed under the sponsorship of Strategic Highway Research Program (SHRP). The SHRP defined HPC for bridge structures by three requirements: namely, a maximum w/c of 0.35, a minimum durability factor for protection against freezing and thawing cycles, and a minimum compressive strength. Under the SHRP program four types of HPC were developed: Very High Early Strength Concrete (14 MPa in 6 hours), High Early Strength (34 MPa in 24 hours), Very High Strength (69 MPa in 28 days), and High Early Strength with Fiber-reinforcement.⁷ Based on SHRP recommendations, the U.S. Federal Highway Administration (FHWA) sponsored a national program of field testing HPC bridge decks in several states. Mehta and Burrows⁸ reported unsatisfactory results from field experience with some HPC bridge decks. According to the authors:

A 1995 report on the condition of 29 bridges in Kansas stated that there was twice as much cracking with 6400 psi (44 MPa) concrete than with 4500 psi (31 MPa) concrete. In 1997, the high-performance concrete deck in the Louetta Overpass – a showcase bridge in Texas – cracked more than the conventional concrete deck in the adjoining lane. In Denver, the high-strength concrete in the 23rd Street Viaduct cracked even before construction was finished. This cracking was due to very high thermal contraction and autogenous shrinkage resulting from the use of a high cement content (w/c = 0.31), and a fast-hydrating Type II cement. The cement fineness was 391 m²/kg and the C₃A-plus-C₃S content was 72%. The cracking tendency of this concrete mixture might have been exacerbated by silica fume, which tends to increase the autogenous shrinkage. In conventional concrete, the autogenous shrinkage of

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less than 50 millionths can be ignored, but a high-strength concrete mixture may have an autogenous shrinkage of several hundred millionths.

In a survey sponsored by National Cooperative Highway Research Program, full-depth transverse cracks with a 1 to 3 m spacing were reported in numerous cast-in-place concrete bridge decks even before the structures were less than one month old.⁹ With massive structures such cracks are attributed to high thermal shrinkage and drying shrinkage of high-strength concrete mixtures, made with high cement content and low w/c. Concrete mixtures of this type exhibit very high early compressive strength (viz. 25-40 MPa in 1-day), and a correspondingly high elastic modulus and reduced creep potential. Consequently, they are prone to cracking at early age.

In the April 1998 issue, the ASCE News—a publication of the American Society of Civil Engineers issued a report card on the state of durability of concrete structures, in general, at the close of the 20th century. This report card assigned a “D” grade, (i.e. very poor grade) to durability. It was reported that a huge amount of money, some 1.3 trillion dollars were needed for repair and replacement.

That concrete progress is not linear, but moves in a zigzag course was demonstrated when FHWA and several other U.S. organizations stopped advocating the use of high-early strength HPC mixtures for cast-in-place, massive, bridge elements. In 1998, a FHWA report contained the following guidelines with regard to HPC for building bridges in the 21st Century¹⁰:

*HPC is a concrete that has been designed to be more durable and, if **necessary**, stronger than conventional concrete. HPC mixtures are composed essentially of the same materials as conventional concrete but the proportions are engineered to provide the strength and durability needed for the structural and environmental requirements of the project.*

The Port Authority of New York and New Jersey maintains many transportation facilities in the New York City metropolitan area, including bridges, tunnels, and airports. The concrete used at these facilities is subjected to high traffic volume and harsh weather conditions. According to Bognacki et al.¹¹:

“In the past, as with other specifications, our concrete specifications only emphasized a compressive strength for a mixture proportion, and did not give the same emphasis to concrete permeability and durability. It is of little consolation to know that the compressive strength met the required contract requirement when the concrete is cracked, spalled, and delaminated. This necessitates significant repair work and causes delays at these transportation facilities, inconveniencing the public.”

Learning from experience, the Port Authority of New York and New Jersey set an example that offers a step in the right direction for concrete durability enhancement. The Agency is now specifying performance-based concrete mixtures, for instance, a

maximum limit of 1500 coulombs chloride permeability at 90 days by ASTM Test Method C1202, and 0.42 – 0.45 w/cm in fresh concrete by AASHTO Microwave Test, T 318-02. The Agency also requires the contractor payments to be made only after strict conformance to the specifications. To make highly durable concrete mixtures, Bognacki et al.¹¹ offer the following advice:

It is very difficult and uneconomic to produce durable concrete for infrastructure work without the use of pozzolans. Concrete mixtures with less than 400 kg/m³ cementitious material including 30 % fly ash or 40% granulated slag were user friendly, economical, and highly crack-resistant.

CONSEQUENCES OF PREMATURE DETERIORATION OF CONCRETE STRUCTURES

One of the consequences of premature deterioration of concrete has been an escalation in the cost of litigation, repair and replacement of numerous structures that have failed to meet the service life expectation. Considering the heavy economic losses, owners and designers of concrete structures have started paying more attention to the concept of life-cycle cost.

There is another important reason why the goal of achieving a radical enhancement in durability of yet-to-be-built concrete structures will have to be pursued with a sense of urgency. Among the manufacturing industries, the concrete industry is the largest consumer of materials in the world. It is estimated that today, globally, we are producing concrete at the yearly rate of some 12,000 million tonnes. The concrete industry therefore requires a very large volume of natural materials like sand, gravel, and crushed stone. Furthermore, it is consuming approximately 1,700 million tonnes/year cementing materials composed mostly of portland-cement clinker – a manufactured product that not only is highly energy-intensive but also responsible for large emissions of carbon dioxide, which is the most voluminous global-warming gas. Portland-cement clinker production also consumes large amounts of limestone and fossil fuels. Due to unsatisfactory durability of a large number of concrete structures, considerable volumes of concrete are being used for repair and replacement of structures. ***In short, the resource productivity of the concrete industry is not high, and the industry is not sustainable at its current global rate of consumption of natural resources and production of green-house gas emissions.***

For sustainable development, an obvious long-term approach is to plan for a gradual reduction in the future concrete consumption rates through radical enhancement in durability¹². Reinforced concrete structures are typically designed for a service life of 40 to 50 years. Extension of service life by 10 to 20 years, or by a factor of two at the most, has been possible by adopting special methods that target a specific cause of potential distress, for instance the use of corrosion-inhibiting admixtures and epoxy-coated steel to mitigate the reinforcement corrosion. If we want to enhance the durability of concrete infrastructure by a factor of 5 or 10, the current methods are obviously inadequate.

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Hawken et al¹³ described a movement, launched in 1994, by the Factor Ten Club which is composed of a group of scientists, economists, and business-people. This group believes that, within 30 years, nations of the world can achieve a ten-fold increase in resource use efficiency, corresponding to a 90% reduction in the consumption of energy and materials. For example, in order to increase the resource efficiency of the concrete industry by factor 10, most of the concrete infrastructure being built today ought to be designed for a service life of 500 years instead of the conventional 50. This is not an unrealistic goal. In fact, as will be discussed in this paper, we do have the science and technology to achieve a radical enhancement in durability of ordinary concrete by a cost-effective method.

A SEARCH FOR THE ROOT CAUSE OF DAMAGE

Field experience with deteriorated concrete structures shows that, *in order of decreasing frequency, the apparent causes of deterioration generally are the corrosion of steel reinforcement, frost action, alkali-silica reaction, and sulfate attack*. In practice, the deterioration of concrete is seldom due to a single cause; at the advanced stages of material's degradation, more than one deleterious process is found at work. Usually the physical and chemical processes of deterioration are so closely intertwined and mutually supporting that separation of cause from effect becomes impossible. If we examine the mechanisms of deterioration separately, they appear to be different from each other. However, irrespective of the apparent cause of deterioration, it is generally observed that *the material failure occurs through a succession of expansion and cracking cycles in which the presence of free water is always necessary*.

It is well known that ordinary concrete mixtures, with moderate cement content (300 to 350 kg/m³) and moderate w/c (0.45 – 0.60), when properly consolidated and cured, are essentially watertight. So, what is the source of free water in hardened concrete? For an answer to this question let us examine the four causes of deterioration of concrete individually:

Frost Action

Commenting on the *damage to concrete by frost action*, Valenta¹⁴ suggested how the material lost its watertightness:

Continuous microcracks, linking into wider cracks originating from the concrete surface, play the greatest role in reducing concrete's impermeability. By facilitating the ingress of water from external sources, they would increase the degree of saturation of concrete, which is a necessary pre-requisite for any damage to be caused by frost action.

Reinforcement Corrosion

To highlight the role of water and microcracks in *the reinforcement-corrosion-related damage* to concrete structures, Mehta and Gerwick¹⁵ described a case history from the

San Francisco Bay Area. In 1980, several 17-year-old, heavily reinforced spandrel beams of the San Mateo-Hayward Bridge had to undergo expensive repairs due to serious cracking that was attributed to corrosion of the embedded steel reinforcement. The beams had been made with a high-quality concrete (370 kg/m³ cement, 0.45 w/c). The damage was confined to the underside and windward faces of beams directly exposed to seawater spray. Interestingly, only the precast, steam-cured beams had been damaged; no cracks and corrosion were observed in the naturally cured, cast-in-place beams made at the same time with the same concrete mixture, and exposed to the same environment of service.

The authors proposed that, as a result of heavy reinforcement and differential cooling rates, microcracks must have occurred in the massive beams (8 by 3.7 by 1.8 m) during the steam-curing process. These pre-existing microcracks later enlarged and became continuous on exposure to more severe weathering action on the windward faces. Thereafter, penetration of the salt water to the surface of the reinforcing steel set the stage for the corrosion-cracking-corrosion type of chain reaction, which led to serious damage. A diagrammatic presentation of the Mehta-Gerwick hypothesis is shown in Fig. 1.

Sulfate Attack

Commenting on the **sulfate-related damage** to concrete sleepers by delayed-ettringite formation, Collepardi⁶ made the following observation:

Previously microcracked beams when exposed to alternate cycles of rain and sunshine were severely macrocracked and damaged; those exposed to rain but kept under shade showed less distress; and those totally protected from rain and direct exposure to sun remained undamaged, without any growth in the microcracks.

Alkali-Silica Reaction

In regard to concrete **damage from alkali-silica reaction**, Swamy¹⁶ made the following comments:

Funny things can then happen in real life – the interior columns of an exposed bridge, sheltered from direct sunlight and rain, may show no cracking whilst the exterior columns may develop extensive cracking. A structural member, when partly sheltered and partly exposed by the nature of the structure, showed very different crack patterns, with extensive cracking in the exposed faces and little or no cracking on the sheltered parts.....

Exclude water - and one can almost have a trouble-free reaction even if concrete contains reactive aggregates and mobile alkalis. Marked deterioration due to the alkali-silica reaction, in field practice, therefore, occurs under wet environmental conditions.

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Thus, with every one of the four causes discussed above, water turns out to be the common cause of concrete damage, and interlinking of macrocracks and voids by growth in microcracks appears to be the source of penetration of free water into hardened concrete.

A HOLISTIC APPROACH TO DETERIORATION OF CONCRETE

In a 1994 paper, the author¹⁷ proposed a holistic approach to concrete deterioration. Unlike the previous reductionistic approaches, the holistic approach is not “cause specific” in the sense that all of the primary causes of concrete deterioration are addressed together. Also, instead of singling out one of the several components of the cement paste or concrete as responsible for the damage, this approach considers the effect of the service environment on all of the components of the material. Furthermore, it takes into consideration the field experience that the degree of water saturation of concrete plays an important role in expansion and cracking mechanisms regardless of other factors that might have contributed to the damage (i.e., frost action, corrosion of reinforcing steel, alkali-aggregate reaction, or sulfate attack).

Among the key features of the holistic approach to concrete deterioration, the point worthy of special attention is that during the first stage, little or no apparent damage to the structure takes place as long as it remains watertight. The second stage marks the initiation of the damage which occurs at a slow rate at first, and then proceeds rather rapidly as the system becomes more permeable. It is suggested that during the latter part of this stage the hydraulic pressure of the pore fluid in saturated concrete rises due to one or more expansive phenomena (e.g., freezing of water, corrosion of the reinforcing steel, and swelling of microcrystalline ettringite or alkali-silica gel). If, at the same time, the hydroxyl ions from the cement paste are being leached away and replaced by acidic ions, such as chloride or sulfate, the calcium silicate hydrate would suffer a loss of adhesion, and as a consequence the concrete strength and elastic modulus are reduced. The damage to the structure accelerates when the microcracks have grown into macrocracks as a result of the two mutually reinforcing processes of damage. *These features of the holistic approach emphasize the common role of microcracks in the initiation and propagation of concrete damage, and provide the basis for the three-stage concrete damage process that will be discussed later.*

MICROCRACKS – THEIR SIGNIFICANCE AND ORIGIN

For a variety of well-known reasons, even with structural members designed for compressive loads, tensile stress and tensile cracking are unavoidable. The reinforcement of concrete with steel does not eliminate tensile cracks but it does restrict the crack-widths to 0.15 mm or less. The fine cracks, known as microcracks, are too small to be visible and quantified, and are therefore ignored in the structural design and construction practice. It is generally accepted that microcracks caused by settlement, plastic shrinkage, restrained thermal shrinkage and drying shrinkage, and accidental overloads would not have any adverse effect on the static behavior of concrete structures. However, due to the important role played by microcracks in determining the

permeability and durability of concrete in service, it is desirable to understand their origin and growth.

Hydration reactions of portland cement minerals produce a multiphase product that consists primarily of an adhesive, poorly crystalline, C-S-H (calcium silicate hydrate) phase, and some well crystalline products including calcium hydroxide. In freshly mixed and compacted concrete, water films forming around the coarse aggregate particles raise the w/c in close proximity to these particles. In the interfacial transition zone between a coarse aggregate particle and cement mortar, the spaces with high w/c become filled with a porous framework of large, plate-like, oriented, and non-adhesive crystals of calcium hydroxide. In conventional concrete mixtures this is a weak area that is highly vulnerable to microcracking. Therefore, the tensile stress generated by differential movement between the cement paste and the aggregate is relieved by the formation of microcracks in the interfacial transition zone. It means that ordinary concrete will contain microcracks in the interfacial transition zone even before a structure is loaded. The amount of microcracking present depends on numerous parameters including the size and grading of aggregate, cement content, water content, degree of consolidation, curing conditions, and humidity and temperature gradients to which fresh concrete has been exposed.

Let us now examine the influence of microcracks on the behavior of concrete in a structure exposed to stress effects from mechanical loading and environmental loading under severe climatic conditions. From a typical strain-stress diagram of a concrete specimen loaded under compression it can be seen that, until about 50 percent of the ultimate stress, there is no significant change in the strain/stress ratio; however, beyond this point the stress/strain curve begins to deviate appreciably from the straight line as increasingly higher strains are recorded with every unit of additional stress. This phenomenon is attributed to increase in the length, width, and number of pre-existing microcracks in the interfacial transition zone and the beginning of some microcracking in the matrix mortar. In the long run, the proliferation and growth of microcracks will have the effect of reducing significantly both the elastic modulus and the tensile strength of concrete.

Also, when the growing microcracks link up with existing macrocracks and voids in concrete, this is the point in time that marks a sudden increase in the permeability. The internal damage begins when the water carrying corrosive ions and gases starts to penetrate more readily into concrete. As described earlier, once concrete becomes saturated and any one of the four above-described expansive phenomena is initiated, the hydraulic pressure in the pore fluid would increase, enabling the microcracks to grow. The end result is a further loss of watertightness of concrete followed by expansion, cracking, spalling, and loss of mass. ***Obviously, for a holistic approach to durability, the microcracks and their growth under service conditions is another common denominator that must be controlled if we want to achieve a radical enhancement in concrete durability. To achieve this objective, we must understand and control the microstructure of concrete.***

STAGES OF THE CONCRETE DAMAGE PROCESS

In 1982, Tuutti¹⁸ presented a schematic diagram (Fig. 2) of the two-stage concrete damage process of reinforcement corrosion. There is no corrosion in the initiation stage during which chloride ions or CO_2 are penetrating into concrete. Once the chloride or CO_2 have reached the surface of the reinforcement, the corrosion process will begin and propagate with time. The end of service life is assumed to have reached when the damage to concrete requires repair or replacement. The durability of concrete can be enhanced by delaying the initiation period through methods such as, the use of corrosion inhibiting admixtures, epoxy-coated reinforcing steel, and coating the surface of concrete. These methods are costly and have not yielded radical enhancements in durability, e.g. by factor 5 or 10.

To achieve a radical enhancement in durability of reinforced concrete structures, I suggest for consideration a three-stage damage process that is graphically presented in Fig. 3. Let us assume that growth of microcracks in the concrete microstructure is a form of “internal damage”, because microcracks provide the bridges that interlink macrocracks and voids. This eventually leads to a breach of water-tightness of concrete, which is a necessary pre-requisite for the subsequent stages of damage. It may be concluded, therefore, that before the stages of “initiation” and “propagation” of damage, there exists a stage of “no damage”, neither external nor internal (i.e. growth of microcracks).

The concept of “no damage” state is useful for achieving a radical enhancement in concrete durability. Clearly, without the growth in microcracks, there would be no increase in the permeability of concrete. Any method that successfully prolongs Stage 1 of the damage process (Fig. 3), would have the effect of delaying exposure of the structure to subsequent stages of damage. Thus, by holding the structure in Stage 1 for a very long period of time, it is possible to achieve a radical enhancement in durability.

MODIFYING THE MICROSTRUCTURE OF CONCRETE

As mentioned earlier, the defects and inhomogeneities present in the microstructure of hydrated cement paste (e.g., capillary voids and oriented layers of crystalline calcium hydroxide) are the primary source of microcracks in concrete. The interfacial transition zone next to the coarse aggregate particles (also next to the steel reinforcement) tends to contain a relatively large proportion of capillary voids and microcracks. Reducing the area of the interfacial transition zone in concrete, and elimination of the defects and inhomogeneities within the hydrated cement paste seem to be the proper tools to control microcracks.

The use of low w/c, as mentioned by the ACI Building Code and other codes for durable concrete, enables a reduction in the volume and size of capillary voids, but this alone is not sufficient to reduce the cement paste content of concrete which is the source of microcracking from thermal shrinkage and drying shrinkage. To reduce the cement paste content, both the water content and the cement content must be reduced as much as

possible. Unfortunately, this is not mandated by building codes because a reduction in the cement content is accompanied by a corresponding reduction in early strength, and that seems to be unacceptable to the strength-driven construction practice of today.

In fully hydrated portland-cement pastes, approximately 24% of the hydration product by mass consists of oriented, heterogeneously distributed, and weakly bonded layers of calcium hydroxide crystals. They debond easily under the influence of tensile stress, and therefore serve as potential site for the formation of microcracks. By transforming all or most of the calcium hydroxide into the calcium silicate hydrate phase (the predominant phase produced by portland cement hydration), a much more homogenous hydration product would result. Thus, concrete mixtures with fewer microcracks can be produced by the use of blended portland cements containing a large proportion of pozzolanic cementitious materials.

The above-described principles for controlling the microstructure and behavior of concrete under most of the exposure conditions encountered in service can be readily incorporated into general construction practice. Obviously, in addition to a reduction in the water content and the cement content, among other microstructure-influencing factors that will have to be taken into consideration in the design of concrete mixtures are the types and amount of mineral admixtures and chemical admixtures, the type and grading of aggregate, and consolidation and curing practice. By using a simple approach to concrete microstructural engineering, it is possible to expect radical enhancement of the service life of concrete infrastructure in a cost effective manner. It is in this context that high-volume fly ash concrete technology, briefly described next, looks most promising.

HIGH-VOLUME FLY ASH CONCRETE

The high-volume fly ash (HVFA) concrete developed by Malhotra¹⁹ represents an emerging technology that shows how highly durable and resource-efficient concrete structures can be built in the future. Laboratory and field experience have shown that fly ash from modern coal-fired power plants, generally characterized by low carbon content and high fineness, when used in a large volume (typically 50-60% by mass of the total cementitious material) is able to impart excellent workability to concrete mixtures at a water content that is 15-20% less than without the fly ash. To obtain adequate strength at early age, further reductions in the mixing water content can be achieved with better aggregate grading and use of a superplasticizing admixture.

According to the author¹², compared to a conventional 25-MPa portland-cement concrete mixture, the corresponding HVFA concrete mixture required one-third less mixing water to obtain similar consistency (125-150 mm slump) and 28-day strength. The total volume of cementitious paste in HVFA concrete was nearly 16% less than the reference portland-cement concrete. As a result, the drying-shrinkage cracking potential of HVFA concrete is much reduced. Similarly, the thermal-shrinkage cracking potential is also greatly reduced because, with 50% cement replacement by mass, typically the HVFA concrete generates nearly 40% less heat of hydration at early age.

Petrographic examination of a HVFA concrete core, obtained from a massive foundation designed for a service life of 1000 years, revealed a very homogeneous microstructure containing essentially no microcracks, capillary voids, and calcium hydroxide crystals. The foundation is now six years old, and to date has shown no cracking whatsoever. Mehta and Langley²⁰ have described the concrete mix design and construction practice for this project. From laboratory test data on corrosion of steel reinforcement, alkali aggregate expansion, sulfate attack, and chloride permeability (ASTM C1202), Malhotra²¹ has confirmed the excellent durability characteristics of HVFA concrete mixtures. Mixture proportions, construction practice, and properties of HVFA concrete used for massive structures in North America and for pavements in India are described in a recent publication²².

CONCLUDING REMARKS

To move toward the goal of a sustainable construction industry, we must achieve a radical enhancement in the durability of portland-cement concrete which is the most widely used material of construction in the world today. This is possible if we critically examine the sequence of deleterious changes in the concrete microstructure that occur when structures are subjected to severe environmental conditions in addition to normal static and dynamic loads. Using a holistic approach, the author has identified the root causes that are common to all of the well-known physical-chemical phenomena responsible for concrete deterioration. This has led to the development of the concept of a three-stage sequence of concrete damage process summarized as follows:

- *A high degree of water saturation is one of the three root causes for durability problems because, in almost every case, it precedes the occurrence of any visible damage to concrete structures. Water is not only the primary vehicle for transport of ions and gases into concrete but also is implicated in expansion and cracking mechanisms in solids.*
- *Microcracks in concrete are a second root cause of durability problems because their growth and connectivity with macrocracks and voids marks the point in time when the stage of "no damage" ends, and the stages of initiation and propagation of damage begin as a result of loss in watertightness. Being discontinuous and invisible to the human eye, microcracks are ignored by the strength-driven design and construction practice in the concrete industry.*
- *The first root cause of the concrete durability problems is the presence of inhomogeneities in the hydrated cement paste (e.g. large capillary voids and oriented layers of crystalline calcium hydroxide). These inhomogeneities serve as potential sites for microcrack formation when concrete experiences tensile stress from mechanical loading and environmental exposure conditions.*
- *By a judicious selection of concrete-making materials and mix-proportions, and by proper construction practice, the microstructural*

inhomogeneities in concrete can be considerably reduced and the durability of structures can be radically enhanced.

Coincidentally, in conformity with the scientific principles of concrete durability enhancement discussed here, a simple and cost-effective concrete technology called the HVFA system has already been developed and field-tested during the last 20 years. Large volumes of fly ash are readily available in many countries of the world, and similar results can be achieved by using, with or without fly ash, a combination of other cement replacement materials, such as metallurgical slags, rice husk ash, natural pozzolans, and pulverized limestone.

There are a number of indicators showing that the era of lack of progress in durability of concrete structures may finally have come to an end.

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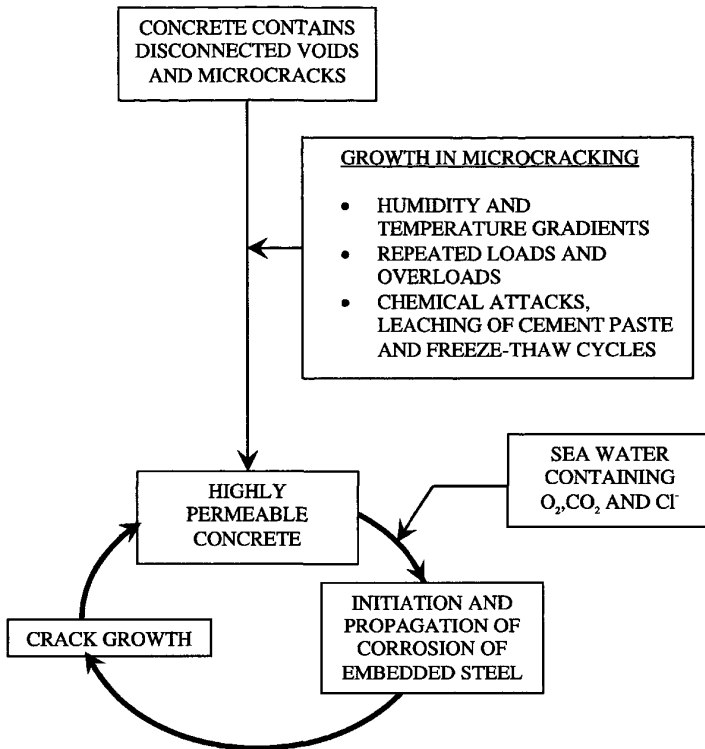


Fig. 1 – Diagrammatic presentation of the cracking-corrosion-cracking cycle in reinforced concrete (Mehta and Gerwick, 1982).

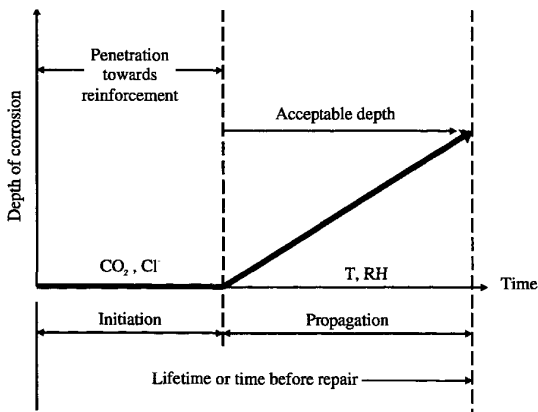


Fig. 2 – Schematic diagram of steel corrosion process in concrete (Tuttle, 1982).

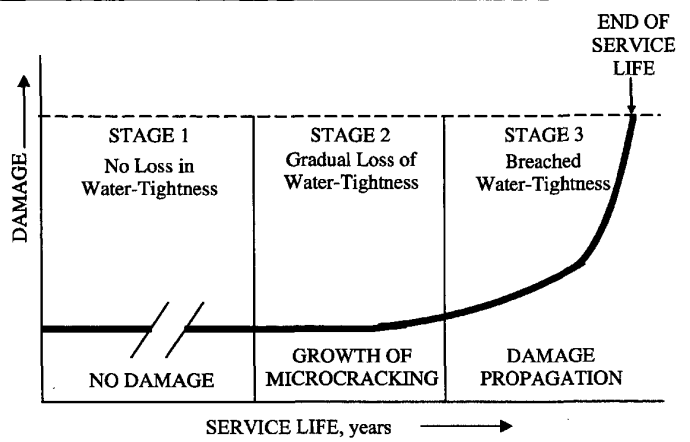


Fig. 3 – A holistic approach illustrating the three stages of concrete damage process.

Characterizing Reinforced Concrete Beams Exposed During 40 Years in a Natural Marine Environment – Presentation of the French Project “Benchmark des Poutres de la Rance”

**by O. Poupard, V. L’Hostis, S. Catinaud, S. Laurens,
I. Petre-Lazar, and A. Raharinaivo**

Synopsis: Several ageing models are available for the prediction of the mechanical consequences of rebar corrosion. They are used for service life prediction of reinforced concrete structures. Concerning corrosion diagnosis of reinforced concrete, some Non Destructive Testing (NDT) tools have been developed, and have been in use for some years. However, these developments require validation on existing concrete structures. The French project “Benchmark des Poutres de la Rance” contributes to these aspects. It has two main objectives: (i) validating of mechanical models to estimate the influence of rebar corrosion on the load bearing capacity loss of a structure, (ii) qualifying the use of the NDT results to collect information on steel corrosion within reinforced-concrete structures. This project started in 2004 for 2 years. Ten French and European institutions from both academic research laboratories and industrial companies are contributing. This paper presents the project that was divided into several work packages: (i) the reinforced concrete beams are characterized from non-destructive testing tools, (ii) the mechanical behavior of the beams are experimentally tested, (iii) complementary laboratory analysis are performed and (iv) numerical simulations will be finally compared to the experimental results obtained with the mechanical tests.

Keywords: benchmark; corrosion; modeling; NDT methods; reinforced concrete

INTRODUCTION

In the context of the civil engineering structures, rebar corrosion is a major cause of damage and early failure of reinforced-concrete structures, and consequently it is recognized as a key issue for ageing concrete structures (1). Initially, reinforcing steel embedded in concrete is naturally protected from corrosion by the high alkalinity of its interstitial solution (1-5). However, this passive film can be destroyed by aggressive ions (chlorides, for instance, or carbonation) (4).

The approaches usually used for designing or rehabilitating civil engineering structures are based on the estimation of the corrosion initiation time as a function of the materials properties. However, in most cases, this initiation time is exceeded, rebar is already corroding, and it is essential to know the influence of corrosion on the mechanical behavior of concrete structures. This knowledge will contribute to determine the residual margin of safety of reinforced-concrete structures to the total failure and so enhance the safety of such structures. Previous studies have shown that the mechanical behavior models are appropriate to evaluate the actual security of concrete structures (6-8). Now, these developments need to be validated on existing concrete structures. Moreover, it is important to take into account the recent progress in the use of non-destructive testing methods. The development of NDT applied on materials and condition assessments of bridges has become a major subject of interest for the structural engineering community.

The French project “Benchmark des poutres de la Rance” contributes to these items through a study of 20 reinforced concrete beams exposed for 40 years in a marine environment.

OBJECTIVES

The two main objectives of this project are:

- To validate existing mechanical models for estimating the influence of rebar corrosion on the load-bearing capacity loss of a structure by taking into account the reduction of steel cross section and/or the decrease of steel-concrete bond,
- To establish the reliability of the use of non-destructive results for collecting information on steel corrosion within reinforced-concrete structures.

Finally, this project will allow:

- *To build up a reference:* by establishing a large database from characterization tests of prestressed beams after 40 years exposure period in a marine environment (mechanical, electrochemical and physico-chemical properties) for improving the knowledge on the influence of corrosion on mechanical performance of reinforced concrete elements (reduction of steel cross section, loss of bond, etc.);
- *To reduce the uncertainties due to simulations and measurements:* by selecting adapted mechanical models for the design or for evaluating the security margins of reinforced concrete structures exposed to corrosion, and by promoting the validation of some future mechanical models based on the experimental database obtained during this project;
- *To increase the development of non destructive methods for inspection and control;*

- *To test the approach based on durability indicators for estimating the performance of concrete in new civil engineering structures.*

PARTNERS

The partners in the “Benchmark des poutres de la Rance” are:

- For the experimental part: Centre d’Expertise de Recherches et d’Etudes du Bâtiment et des Travaux Publics (CEBTP, France), Electricité de France (EDF R&D, France), Commissariat à l’Energie Atomique (CEA, Saclay, France), Laboratoire Central des Ponts et Chaussées (LCPC, Paris), Laboratoire Matériaux et Durabilité des Constructions (LMDC, Toulouse), Laboratoire d’Etudes et de Recherches sur les Matériaux (LERM, France), OXAND (France), SP institute (Sweden).
- For the modelling part: EDF R&D (France), CEA Saclay (France), LCPC (Paris, France), LMDC (Toulouse, France), OXAND (France), Laboratoire de Mécanique et Technologies (LMT, France), Laboratoire de Mécanique de Lille (LML, France), Laboratoire GEM (Nantes, France), Institute Eduardo Torroja (Madrid, Spain).

DESCRIPTION OF THE BEAMS

This project is based on a long-term experimental program started in 1962 by the “Union Technique Interprofessionnelle des Fédérations Nationales du Bâtiment et des Travaux Publics”. Initially 40 prestressed concrete beams were cast and stored in marine environment (Rance dam) (Figure 1). In 1976, these beams were moved to the Sainte Anne de Portzic harbour (IFREMER, Brest), in a tidal zone. Finally, since 2000, 20 beams have been stored on a land site (CEBTP, Saint Rémy les Chevreuse). The dimensions of 20 beams are 2500×200×200 mm. The ends (on 250 mm) are protected by a bituminous binder. Each beam is identified by three numbers: the first one corresponds to the lay-out configuration (see Figure 2 for more details), the second one to the granulometry of concrete mixture and the last one to the cement content (Table 1). Figure 2 illustrates the layout details of the beams. The beams are reinforced with passive plain carbon steel bars ($\phi 6$ mm) and 10 stirrups ($\phi 6$ mm) spaced at about 250 mm. The concrete cover is 16 or 41 mm, depending on layout configurations. For each type of beams, 4 concrete designs are studied (Table 1): two different cement mixtures (300 and 400 kg/m³ of concrete) and two gradings (continuous and discontinuous). The concrete is prestressed with wires, 7 mm in diameter, placed under a plastic sheath, 12 mm in diameter, and anchored at the beam ends. For beams of Types 1 and 2, prestressing cables are centered along the axis of the beam. For Types 4, 6 and 9, they are in the lower part of the beam, as shown on Figure 2. So, in these cases, the concrete prestressing reproduces cracking pattern similar to that in a structure during a long-term exposure. Table 2 presents the initial longitudinal stresses within the concrete (in upper and lower parts of the beam, respectively) due to presence of prestressing cables.

METHODOLOGY AND DETAILED WORK PROGRAM

This project is structured into seven steps (Figure3):

- ❑ **Step 1** - A survey of structural damage is performed from non-destructive testing methods;

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- ❑ **Step 2** - The global mechanical behavior of the reinforced beams is experimentally estimated by one partner (CEBTP). The results will be kept secret until step 5.
- ❑ **Step 3** - Complementary laboratory analysis are carried out for characterizing passive reinforcing steels, prestressing cables, corrosion products and concrete. These data are transmitted as input data to the modeling teams;
- ❑ **Step 4** - Modeling teams apply their mechanical models to the global and local mechanical behavior of the beams, the input data are the experimental results obtained during steps 1 and 3;
- ❑ **Step 5** - Numerical simulations results (Step 4) are compared to the corresponding experimental results (Step 2).

Conclusions are focused on the validity of the different mechanical models, and on the reliability of the non destructive methods. Each step is detailed below (partners are indicated in bold letters).

Characterization from non-destructive testing methods (Step 1)

Visual inspection

First, all the beams are visually observed. It aims to survey the beams in order to detect which type of structural damage is being developed in the structures. Regarding reinforced corrosion problems, three types of visual inspection results are given:

- Maps of cracking positions: cracks due to reinforcement corrosion are drawn taking into account the position of the visible cracks and their widths,
- Corrosion maps: the locations of the rusted areas on the beams are drawn,
- Information on spalling, delamination and loss of concrete cover are also reported.

Reinforcement geometry

The reinforcement details is obtained from magnetic measurements¹ to control:

- concrete cover thickness,
- rebar position : transverse and longitudinal reinforcing steel bars.

Electrochemical methods

Electrochemical measurements are performed on the beams to evaluate in a non-destructive way the rebar corrosion activity. First, half-cell potentials are measured. The principle of this method is based on the measurement of the potential difference between rebars and a reference electrode. A multi-electrode device² is used to collect data (three measurement lines per side and one reading per cm along the beam length). The reference electrode is a copper/saturated copper sulfate (CSE = +318mV/NHE) electrode. Measurements are performed according to RILEM recommendations (9).

¹ Ferroskan®, Pachometer

² Canin®

The measurement of half-cell potential is a qualitative way to mainly locate corroding. To characterize in a quantitative way the corrosion activity, it is essential to evaluate the steel corrosion rate. The corrosion rate, v_{corr} , is deduced from the polarization resistance (R_p) measurement (10). The current density, I_{corr} , is then obtained from Stern and Geary equation (Eq. 1) (11):

$$I_{corr} = \frac{B}{R_p} \quad (\text{Eq. 1})$$

with B a constant parameter which varies from 13 to 52mV depending on the studied system. In this study, B is considered as equal to 26mV (if R_p and I_{corr} are expressed respectively in $k\Omega$ and μA), according to Rilem recommendations (10).

The corrosion rate can be deduced from corrosion current from Faraday equation. If corrosion current is expressed in μA , corrosion rate (in $\mu m \cdot year^{-1}$) is given by the relation (Eq. 2) assuming uniform corrosion across the rebar surfaces:

$$V_{corr} = 11.6 \frac{I_{corr}}{A} \quad (\text{Eq. 2})$$

where A is the steel surface area (in cm^2).

Table 3 summarizes the different techniques for corrosion rate and resistivity measurements used in this project.

Electromagnetic methods: Radar

The general principle of radar sounding of civil engineering structures, based on the propagation of EM impulses, has been reported by many authors (12,13). A radar system³ was used in association with two high frequency ground-coupled 1.5 GHz antennas.

An original approach proposed by the « Diagnosis » group from the LMDC laboratory consists in using radar technology for the prediction of pathologic risks affecting concrete structures. For example, the electromagnetic properties of concrete, that govern radar wave propagation, are influenced by moisture and/or chloride content. Thus, radar measurement is sensitive to the physical condition leading to corrosion in reinforced concrete.

Previous laboratory studies (14) highlight that the energy directly radiated by the transmitter towards the receiver, named the direct wave (DW) or direct signal, can provide reliable data on the coupling material properties. In this study, a simple interpretation criterion based on the DW positive peak amplitude was tested as first approach :

- Small DW amplitude (high moisture and/or chloride content) = High corrosion risk,
- High DW amplitude (low moisture and/or chloride content) = Low corrosion risk.

³ A SIR-2000 GPR developed by Geophysical Survey System Inc. (GSSI®). SIR-2000 is a GPR data acquisition system and the GSSI 1.5 GHz antennas provide high quality waveforms allowing simpler signal processing.

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Characterization of the constitutive materials (reinforcing steel, prestressed cables, corrosion products and concrete) (Step 3)

After the non-destructive inspection of the beams, 10 beams are used to characterize the materials properties and to estimate the aggressive species penetration (chloride front and carbonation depth). The carbonation depths are measured by a phenolphthalein test. For this purpose, freshly broken concrete surface are sprayed with a phenolphthalein solution at various locations along the beam. Sound concrete appears red while the color of the carbonated concrete remains unchanged. This change occurs in the pH range 8.3-10. Powdered concrete samples are taken for determining contents of "total" and "free" chloride and estimating chloride penetration profiles at different locations on the beams according AFPC-AFREM recommendations (15).

Samples are collected for each concrete type to characterize the material microstructure features (porosity, pore size) which are in strong relationship with transport properties. MIP tests and water absorption measurements are performed.

Cores are also taken from specimens to assess the mechanical properties of concrete at 40 years (compressive strength, elastic modulus and tensile strength).

Mechanical measurements are also performed to estimate the mechanical properties of prestressing cables and passive reinforcements. Tests are also performed to evaluate the residual prestressed value within beams

The nature of corrosion products is well identified, with scanning electron microscope observations, X Ray diffractometry analysis and μ -Raman method. The localized corrosion (pits) areas and depth are also measured, as well as the section loss due to generalized corrosion. Finally, the weight loss of the steel is also determined.

Experimental characterization of the global mechanical behaviour of the beams (Step 2)

10 beams are used to perform mechanical tests on reinforced beams to estimate their global and local mechanical residual behavior. The mechanical tests are: (i) tensile tests and (ii) 4-point bending tests. For these two tests, loads are monotonic and cyclic. The reinforced concrete beams are instrumented with extensometers, displacement sensors, and gauges, to monitor the mechanical behavior during test. It deals with measuring load, displacement and strain.

Tensile test

Figure 4 shows a detail of instrumentation for tensile tests and a photograph of the system layout. This test is only performed on beams type 1 and 2 (where prestressing cables are centered in beam). The experimental results are used for determining the global and local behavior of the beams. For the global behavior, the load-displacement curve is recorded and the ultimate tensile strength is determined. For the local behavior, the concrete local strains (curvatures and mean deformations in concrete) in non-cracked and cracked cross-sections are determined.

Four-point bending test

Figure 5 shows a detail of instrumentation for the 4-point bending tests and a photograph of the system device. Beams are simply supported and two symmetrical loads are applied at a distance of 400 mm from the middle of the beam.

As for tensile tests, the experimental results are requested for the global and local behaviour of the beams. For the global behavior, the load-displacement curve is recorded and the ultimate tensile strength is determined. Vertical displacements field sensors are put along beams to observe displacements at various locations. For the local behavior, the concrete local strains (curvatures and mean deformations of concrete) are determined in non-cracked and cracked cross-sections.

At the end of these mechanical tests, maps are drawn showing the locations of cracks.

The results from this task are being kept secret, and they will be disclosed only at the end of the project for a comparison with simulation model results.

Prediction of the mechanical behavior (Step 4)

The experimental results obtained in Steps 1 and 3 are transmitted to the modeling teams, and a blind prediction is carried out. Simulations are performed with different numerical models to assess the global and local mechanical behavior of the corroded-reinforced concrete. Deterministic and probabilistic models are tested in this project. Simulations are considering no corrosion and actual corrosion damage in beams.

Comparison between experimental and numerical simulations (step 5)

At the end of the project, the experimental mechanical results and the simulations from the models are compared in order to assess the domain of validity of tested models.

RESULTS AND DISCUSSION

This paragraph gives some results obtained during steps 1 and 3 of the project. These are focussed on the visual observation of beams, on the electrochemical measurements and on a synthesis of NDT results, as compared to the real corrosion state of the rebars.

Chloride profiles and carbonation depth

Chemical tests on drilled concrete cores (for chloride profiles) and freshly broken concrete for (carbonation measurements) showed that the four concretes were highly contaminated by chlorides but were not carbonated. For all the tests, the maximum carbonation depth is 1 mm. A characteristic chloride profile of a concrete structure exposed in tidal marine environment is observed with a maximum value at about 10 mm from the surface and a slight decrease within the bulk of the structure. At reinforcing bars depth, the measured values exceed the threshold limit (0.5-1.5%) generally pointed out in literature (16). These results highlight that active corrosion due to chloride ingress can take place at a rebar surface.

Visual observations

Before removing concrete cover, a visual observation gives a global overview of the corrosion state of prestressed concrete beams after 40 years in marine environment. Figure 6 presents crack maps for beams 122 and 622.

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Except beam 912, all the beams show longitudinal cracks due to corrosion attack. They are often associated with some spots of rust stains. For beams type 1 and 2, corrosion appears mainly on lateral sides due to a thin concrete cover (16mm), and little corrosion is observed on the upper and lower sides (41mm). For beams type 4, 6 and 9, cracks are mainly observed on sides and on upper parts. When they exist, longitudinal cracks on lateral sides are mainly located on the upper part, that corresponds to tensioned concrete. On the contrary, the compressive area (lower part) shows few corrosion induced cracks. For some beams type 9, even if concrete cover is thick (41mm), important corrosion appears on upper part.

After removing concrete cover, the actual corrosion damage of each rebar is observed. As expected in chloride contaminated concrete, corrosion occurs in localized areas (pitting attack). For beams type 1 and 2, similar corrosion damage is observed between upper and lower rebars. This can easily be explained by the fact that similar loads are applied in concrete (in upper and part). For beams type 4, 6 and 9, the details of corroded areas show corrosion initiation along the tensile rebars (upper part). Except for some cases, rebars under compression (lower part) in these beams are weakly corroded. It is observed that the highest corrosion risk is in the upper face of type 6 beams with a thin concrete cover of 16mm and prestressing cables highly eccentric. This result agrees with the paper by Francois and Arligue (17). Microcracks in tensioned concrete enhance chloride and oxygen penetration into concrete and damage at steel/concrete interface.

There are large differences in corrosion behaviour depending of concrete mixture. For a given layout configuration, concrete of type 1.2 (continuous granulometry, cement content 400 kg per m³ of concrete) always shows the lowest percentage of corroded areas. Conversely, beams with low cement content (300kg. per m³ of concrete) show the worst behaviour towards corrosion risk. This is in good agreement with previous studies that have shown that the W/C has a major role on the corrosion process within reinforced concrete (18, 19). When this parameter increases, porosity also increases (see Table 1). So aggressive species (chloride, oxygen) can more easily penetrate into concrete and develop high-level corrosion at a rebar surface.

Corrosion rates measurements

First, a synthesis of corrosion rates measurments (from Gecor6[®] device) is presented and allows pointing out some general information. Then, the effect of different parameters (prestressing effect, concrete design) on instantaneous corrosion rates can be studied. Figure 7 presents a review of corrosion potential values and corrosion rates values (in $\mu\text{A}.\text{cm}^2$) measured on all beams. On the same graph, the corrosion defined for corrosion levels (10) are shown. The following observations may be made from the above graph:

- No direct relation is pointed out between the half-cell potential and the instantaneous corrosion rates (i_{corr}),
- Rebars from beams type 1 and 2 (centered prestressing) present corrosion rate values from low to moderate.

- Corrosion rate values measured on the lower face of type 4 and 6 beams (prestressed in their lower part) show a negligible or low corrosion activity.
- On the contrary, the upper faces of these beams present more important corrosion rate values, and the corrosion activity is moderate to high for type 6 beams.
- An important scatter is observed for dispersion corrosion rate values in type 9 beams.

After this global synthesis, remarks can be made on several parameters influencing corrosion rates. Figure 8 presents the corrosion rates values measured for the beams type 4 and 6 (prestressed on the lower part of the beam, and concrete cover of 16mm on the 4 faces). Figure 9a shows the results for the concrete beams containing 300 kg cement per m^3 of concrete, whereas the Figure 9b presents results obtained for the concrete containing 400 kg cement per m^3 of concrete.

For the two cases, graphs point out an important scatter of corrosion activity in the considered concrete zone. The measured corrosion rate is globally more important in the points located on the upper face (tensile area) than in points located on the lower part of the beam (lower face, compressive area). So, the corrosion rates are different in tensioned and in compressed -faces of a beam, but this is not the case for half-cell potential values. These results totally agree with the visual observation presented previously.

Figure 9 presents the influence of concrete design (cement content) on the corrosion activity. Figure 9(a) shows the corrosion rates and half-cell potential results of types 1 and 2 beams centrally prestressed, concrete cover 41mm), whereas Figure 9(b) presents these values for the types 4 and 6 beams (prestressed in the lower part, concrete cover 16mm). Moreover, for beams prestressed in their lower part, only measurements of the upper face are reported (tensioned areas).

The data of the beams type 1 and 2 (Figure 9(a)) clearly show that:

- Concrete with high content cement (400 kg) and a continue granulometry presents the lowest corrosion activities: these conclusions are drawn from corrosion rates values and from half-cell potentials measurements.
- Concretes with low content cement (300 kg) present highest corrosion rates and most negative half-cell potentials.

A similar analysis performed on type 4 and 6 beams (Figure 9(b)) does not point out any effect of the concrete design on corrosion activities. There, the cement content and the granulometry are not the determining parameters on the corrosion behaviour of beams.

Syntheses of some NDT results

Only results obtained on upper and lower faces will be considered in this synthesis. NDT factors (data from visual observations, potential, corrosion rate and radar measurements) measured on upper and lower sides for each beam are presented in Table 4. The discussion is focused on comparing the various factors determined from non-destructive tools and the actual state of rebars.

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From potential measurements, two factors have been estimated: a potential factor and a gradient potential factor. ASTM C876-87 (20) provides a classification for the interpretation of probability of corrosion activity from absolute values. The “potential factor” is defined as the percentage of potential values lesser than the critical threshold – 350 mV/CSE (95% corrosion probability). However it is widely known in literature that interpretation based on this absolute value can result in unreliable predictions of corrosion risk. Indeed half-cell potential value can be affected by various parameters such oxygen concentration at steel surface, concrete cover, etc... (9,21,22). The “potential factor” values presented in Table 4 confirm the influence of concrete characteristics (quality of concrete, cover) and no correlation can be established with the real corrosion state. The use of gradient potential (spatial variation) seems to be more pertinent to evaluate in a qualitative way the corrosion degree within reinforced concrete (14). From half-cell potential database obtained from measurements on beams, a spatial potential gradient is estimated for each spatial location. The “gradient potential factor” is defined as the percentage of gradient potential values higher than a critical threshold. In this study, a critical threshold of 8 mV.cm⁻¹ will be considered.

The following observations may be made from the data in Table 4:

- for types 4, 6, and 9 beams, radar factor values are systematically lower in upper sides.
- for these same beams, electrochemical factor values are higher in the upper sides.
- except for beam 412, type 1.2 concrete generally gives higher values for radar factor and resistivity and lower values for electrochemical factors.

Figure10 illustrates the correlation between these factors and the percentage of corroded areas. On these graphs are distinguished values estimated on lower and upper sides for each beam.

Except for “crack” factor, a good accordance is observed between corrosion diagnosis from NDT factors and corroding rebar area. As expected, correlation is observed for electrochemical parameters (gradient potential and corrosion rate). Concerning radar results, a correlation is found between the DW amplitude value and the corroded areas at rebar surface for a beam side. That is in good agreement with previous laboratory results (15): DW amplitude decreases with increasing percentage of corroded areas. The limitations of “crack” factor is easily explained: longitudinal cracks enhances chloride penetration at rebar surface. For types 1 and 2 beams, the concrete cover is thicker and chloride penetration is not sufficient to induce crack damage at surface concrete. No crack appears before that chloride content at steel surface reaches a threshold. It is interesting to notice that in the case of a result non-expected, NDT indicators confirm actual corrosion risk. For example, due to its concrete design (low W/C), beam 412 should show a good corrosion behaviour. However a high corrosion level is visually observed in upper part of this beam compared to the others beams type 4. NDT indicators values measured on the upper face of this beam are in good agreement with the real corrosion state (high potential gradient and corrosion rate values and low DW amplitude). Finally, in the case of beams of type 4, 6 and 9, NDT factors confirm the difference of activity, for a same beam, in the upper and lower parts. As confirmed

observations after removing concrete, higher corrosion activities are observed for these beams in tensile rebars (upper part).

CONCLUDING REMARKS

The French national project “Benchmark des poutres de la Rance” has been proposed to evaluate the influence of corrosion on mechanical behavior of reinforced concrete structures after a long-term exposure in marine environment. At this time, only results from Tasks 1 and 3 are presented, because the project is not yet fully achieved. The final results and a scientific report will be published at the end of 2005. The information about these beams will be structured as follows:

- A database characterizing their initial state.
- A database characterizing their corrosion state after 40 years in a marine environment (mechanical, electrochemical and physico-chemical properties).
- The comparison between the corrosion diagnosis performed from non destructive testing tools and actual corrosion damage (destructive inspection).
- The comparison between numerical simulations (from various mechanical models) and the experimentally observed behavior.

Concerning the corrosion diagnosis on 16 prestressed concrete beams after 40 years exposure in marine environment, the results are summarized, as follows:

- physico-chemical measurements have confirmed that chloride ions are the only responsible of corrosion attack.
- crack maps gives a global overview of the corrosion area (but only when the corrosion process reaches a critical issue).the tensile/compressive stresses within concrete structure have a significant effect on corrosion of passive reinforcing steel. Conclusion on an “active” corrosion in the tensile part of the beam and a “low” corrosion in the compressive part can be made with confidence.
- the thickness of concrete cover and W/C are important factors determining the ability of aggressive species (chloride and oxygen in our study case) to reach the rebar surface and to start the corrosion process.
- the NDT indicators proposed in this study are well-correlated with the real corrosion state of rebars within concrete structures and will have to be of a first relevance to establish a diagnostic of corrosion.

These initial results are encouraging to support confidence in such NDT tools (radar, half-cell potential (using spatial gradient values) and corrosion rate measurements) to control and evaluate in a semi quantitative way (for electrochemical indicators at least) the corrosion risk in a real reinforced-concrete structure.

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Table 1 – Concrete mixture proportions

Concrete	Total water (l)	mixture proportions (kg per m ³ of concrete)				W/C	Porosity	
		Cement	Sand 0mm/5mm	Gravel 5mm/15mm	Gravel 10mm/25mm		MIP (%)	Water absorption (%)
1.1	210	300	800	320	930	0.7	15.3	15.7
1.2	210	400	550	290	930	0.53	13.1	13.4
2.1	220	300	500	/	1350	0.73	15.7	16.7
2.2	250	400	450	/	1350	0.63	14.5	16.4



Table 2 – Stress in prestressed concrete beams

Beam Type	Stress in concrete (in MPa)	
	Upper part	Lower part
Type 1	1.66 – 1.74	0.73 – 0.77
Type 2	2.63 – 2.95	1.67 – 1.99
Type 4	-0.73 – -0.76	5.46 – 5.77
Type 6	-2.19 – 2.39	6.77 – 7.38
Type 9	-5.07 – -5.30	14.75 – 15.35

Table 3 – Techniques used for electrochemical measurements

	Institution	Techniques	Device
Corrosion rate	LREP	Potentiostatic pulse	Gecor6®
	OXAND/LERM	Galvanostatic pulse	GalvaPulse®
	SP institute	Galvanostatic pulse	Rapicor®
	EDF R&D	Linear polarisation	Gamry®
	CEA/SCCME/LECBA	Linear polarisation	Gamry®
Resistivity	LREP		Gecor6®
	SP institute		Rapicor®

Table 4 – Numerical values of NDT factors and percentage of rebar corroded areas for each beam

Designation beam	Crack factor ¹				Potential factor ² Potential gradient factor ³				Corrosion rate				RADAR factor (arbitrary unity)				Percentage of corroded areas (in %)			
	In upper face	In lower face	In upper face	In lower face	In upper face	In lower face	In upper face	In lower face	In upper face	In lower face	In upper face	In lower face	In upper face	In lower face	In upper face	In lower face	In upper face	In lower face	In upper face	In lower face
Type 1																				
121	0.0	0.0	99.7	98.3	11.0	9.6	c	c	6.5	8742	9124	39.3	13.6							
122	2.4	0.0	100.0	100.0	3.8	4.0	c	c	3.0	7181	8267	31.6	13.3							
Type 2																				
211	0.0	0.0	99.5	100.0	17.9	5.8	c	c	2.9	7708	7808	30.3	28.9							
212	0.0	0.0	8.5	24.7	9.5	17.4	c	c	1.5	10828	9894	8.1	5.5							
Type 4																				
411	9.3	0.0	84.4	100.0	15.9	5.0	2.5	1.3	8042	8185	28.3	22.1								
412	21.4	0.0	71.0	76.8	30.3	3.3	8.5	2.1	8112	10129	36.9	2.0								
421	0.0	1.6	94.2	96.3	20.8	8.8	8.5	2.1	8442	9240	a	a								
422	13.5	12.0	77.6	100.0	35.3	4.5	12.8	3.2	7183	8167	16.8	12.0								
Type 6																				
611	10.1	0.0	75.8	90.7	44.1	5.5	17.1	2.4	8179	8164	84.4	6.3								
612	31.2	0.0	64.3	32.5	15.6	14.8	6.0	0.9	9556	9855	38.9	2.1								
621	7.9	0.0	84.7	95.4	24.6	11.1	11.3	4.7	7349	8810	55.2	9.2								
622	52.9	0.0	69.0	14.3	20.5	5.8	5.6	4.6	8723	9329	50.0	14.6								
Type 9																				
911	21.9	18.3	100.0	98.2	8.7	9.3	7.8	6.2	8563	9288	24.2	b								
912	0.0	0	0.0	0.0	5.0	2.5	3.8	4.4	9689	9896	5.9	b								
921	0.0	17.8	99.3	99.8	5.4	7.2	3.5	0.6	8801	9132	9.4	b								
922	88.5	0	80.4	50.7	16.5	4.2	8.2	0.9	7948	8857	36.9	b								

¹ crack factor = $\sum_{i=1}^N \text{length (i)} \times \text{width (i)}$ where N is the number of cracks existing on the considered face of the beam.

² percentage of potential values lesser than the critical threshold -350 mV/CSE (according ASTM)

³ percentage of gradient potential values higher to a critical threshold. In this study, a critical threshold of 8 mV/cm will be considered.

^a data not yet available

^b no passive rebars in lower face

^c data not measured



Fig. 1 –Prestressed beams during their long-term exposure in marine environment

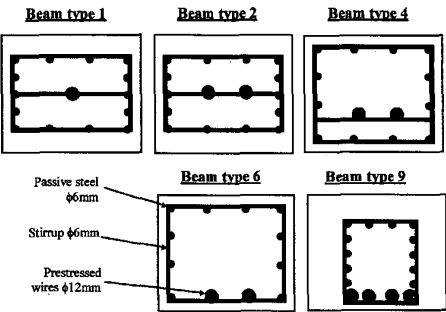


Fig. 2 – Schematic view of the layout details for each beam type

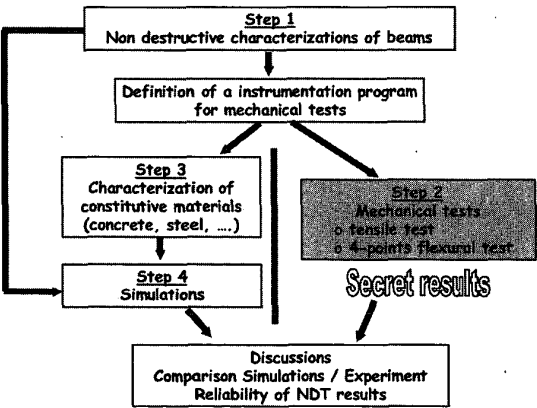


Fig. 3 – Methodology of the benchmark

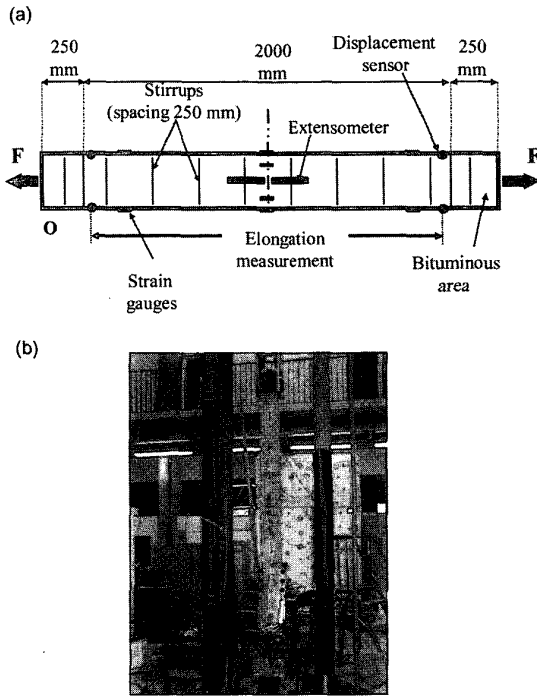


Fig. 4 – Tensile test (a) schematic view of the instrumentation (b) experimental setup

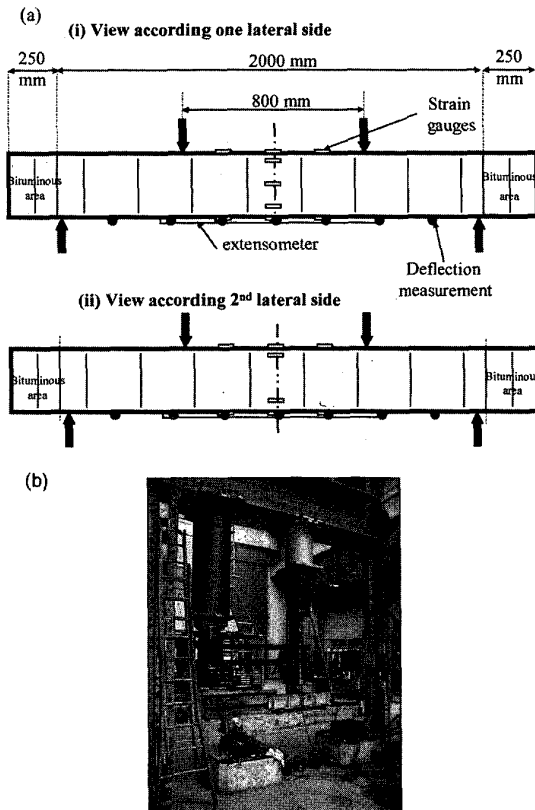


Fig. 5 – 4-point bending test (a) schematic view of the instrumentation
(b) experimental setup

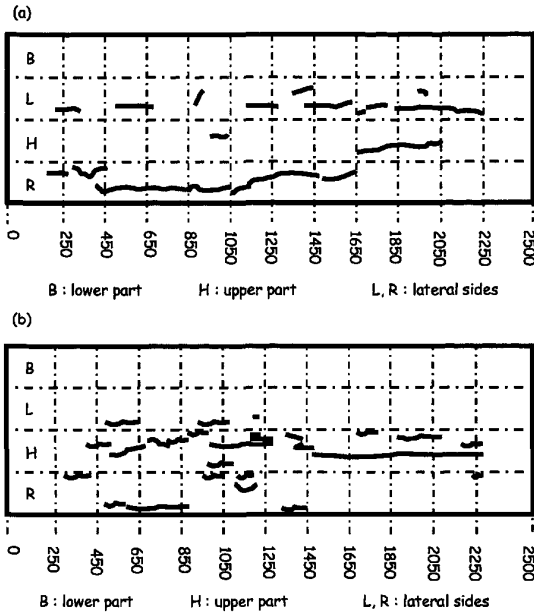


Fig. 6 – Some examples of corrosion-induced crack maps (a) Beam 122 (b) Beam 622

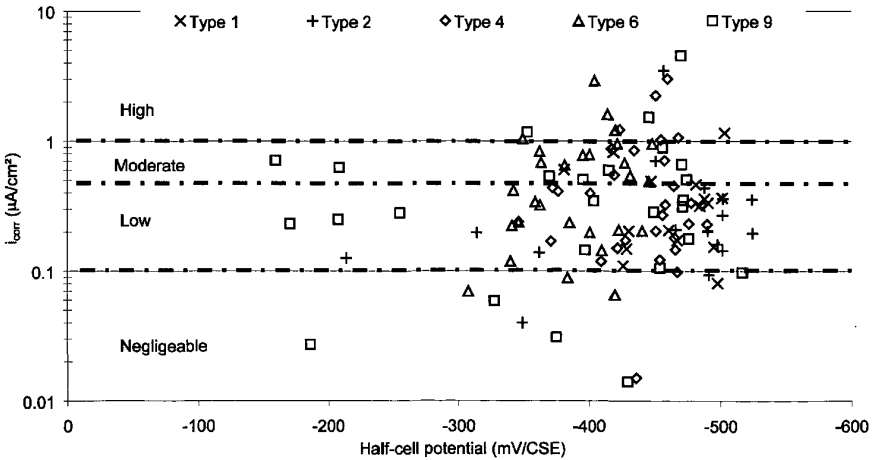


Fig. 7 – Corrosion rates measured on different locations on 20 beams

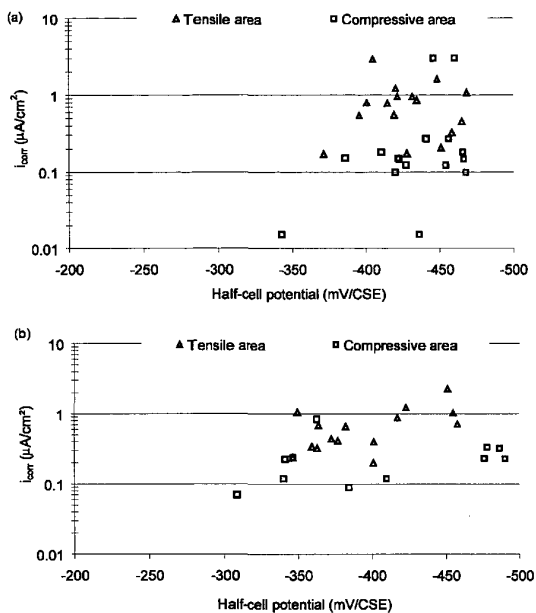


Fig. 8 – Influence of stress in concrete (tensile/compressive areas) on corrosion rate — cement contents (per m³ of concrete) (a) 300 kg (b) 400 kg

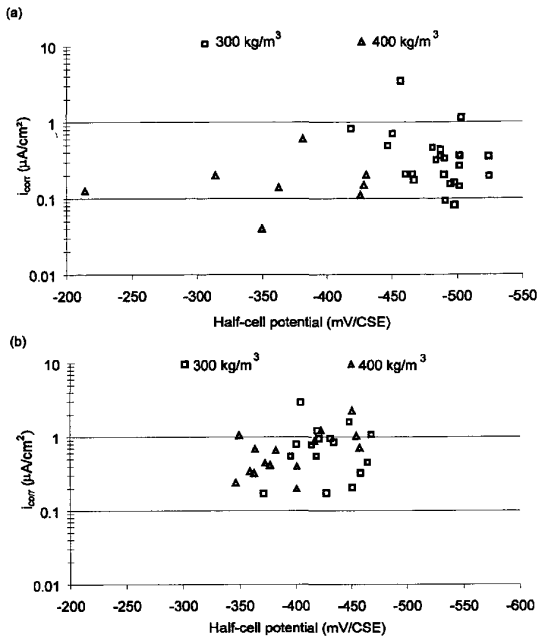


Fig. 9 – Influence of cement content on corrosion rates – concrete cover
(a) 41 mm (b) 16 mm