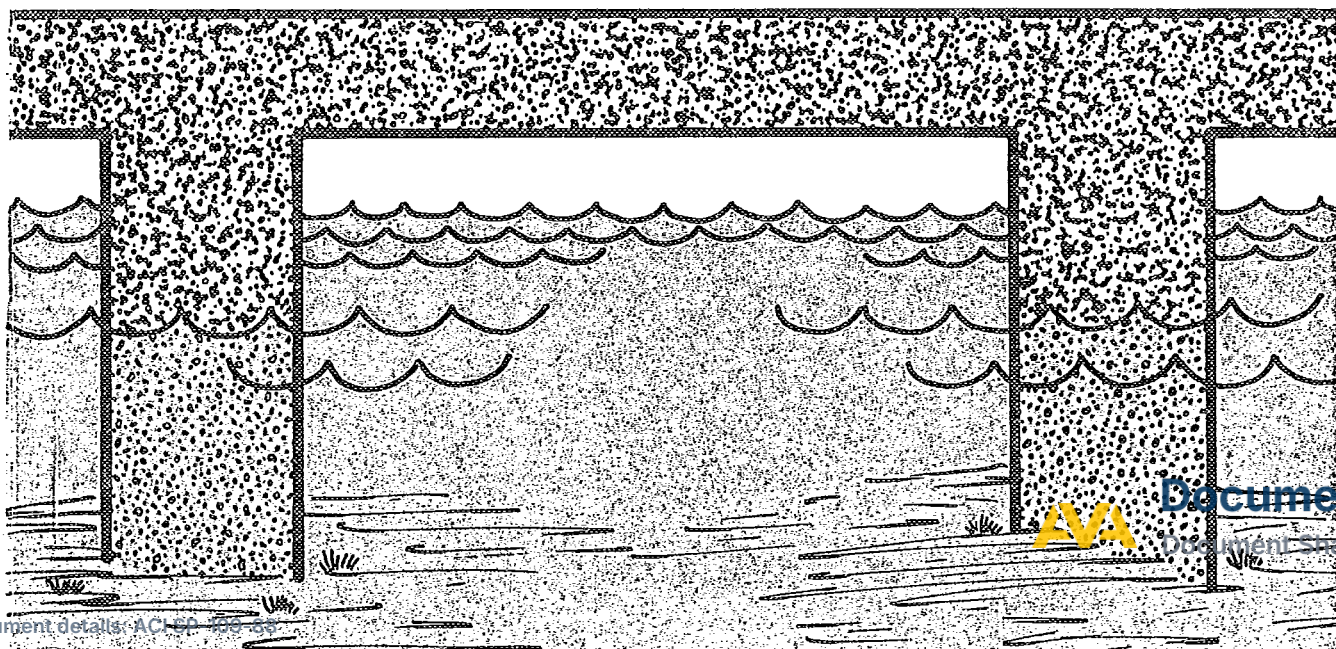


# Concrete in Marine Environment \_\_\_\_\_ Proceedings \_\_\_\_\_ Second International Conference St. Andrews by-the-Sea, Canada 1988

V.M. Malhotra  
Editor

SP-109



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Concrete in Marine Environment  
\_\_\_\_ Proceedings \_\_\_\_\_  
Second International Conference  
St. Andrews by-the-Sea, Canada  
1988

SP-109

V.M. Malhotra  
Editor

American Concrete Institute, Detroit



## PREFACE

The Canada Centre for Mineral and Energy Technology (CANMET) of Energy, Mines and Resources Canada, Ottawa, has played a significant role in Canada for over 20 years in research on durability of concrete. In April 1980, CANMET in association with the American Concrete Institute, the U.S. Army Corps of Engineers, the University of New Brunswick and several other professional organizations sponsored a five-day International Conference at St. Andrews by-the-Sea, New Brunswick, Canada, on the Performance of Concrete in Marine Environment. The purpose of the conference was to bring together representatives of the cement and concrete industries to exchange information on the effects of marine environment, on plain and reinforced concrete, and to explore new areas of needed research and development. The proceedings of the conference containing thirty-three papers from more than ten countries were published as ACI Special Publication SP-65.

In 1988, CANMET in association with the American Concrete Institute, University of New Brunswick, U.S. Army Corps of Engineers, The Concrete Institute and Institute for Research in Construction (National Research Council), sponsored the Second International Conference on the subject of Performance of Concrete in Marine Environment. The conference was held on August 21-26, 1988, at St. Andrews by-the-Sea, New Brunswick.

The conference opened with a paper by P. Kumar Mehta, "Durability of Concrete Exposed to Marine Environment — A Fresh Look" which is followed by a paper dealing with the current status of CANMET's studies on the durability in marine environment of concrete containing supplementary cementing materials. These are followed by papers which deal with durability and corrosion of reinforced concrete and case histories of the performance of concrete in marine environment. These, in turn, are followed by a series of papers on repairs to concrete structures. The last papers deal with the design and construction of reinforced concrete structures in marine environment.

Thanks are extended to those ACI members and others who helped review manuscripts. Particular thanks are extended to Dr. V.S. Ramachandran for his assistance in carrying out extensive editing of several of the submissions.

The help and assistance of Joseph F. Lamond and Bryant Mather of the U.S. Army Corps of Engineers is gratefully acknowledged in making it possible for conference participants to visit the Natural Weathering Exposure Station at Treat Island, Maine.

V.M. Malhotra, Editor  
Chairman, Second International  
Conference on Performance of  
Concrete in Marine Environment

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# Durability of Concrete Exposed to Marine Environment—A Fresh Look

by P.K. Mehta

**Synopsis:** From recently reported case histories of concrete deterioration in seawater, the author has taken a fresh look at the conclusion presented on this subject at the last CANMET/ACI Conference on the Performance of Concrete in Marine Environment. It is confirmed again that between seawater and the constituents of hydrated cement paste harmful chemical reactions, such as carbonation, sulfate attack, and magnesium ion attack can be limited to the surface when well known measures to assure low permeability of concrete have been put into practice.

From standpoint of permeability of concrete the topics discussed in detail include selection of materials and mixture specifications, concreting practice, and control of in-service cracking due to thermal gradients, frost attack, improper loading conditions, fatigue, and corrosion of the embedded steel in concrete. The ACI and FIP Recommended Practice for offshore concrete structures, and recent field experience from the North Sea are compared, to highlight the issues that are relevant to long-time durability of concrete. Since high-strength concretes (50 to 70 MPa), containing water-reducing and mineral admixtures are relatively impermeable, in the opinion of the author they offer an excellent solution to the problem of durability of concrete in seawater environment.

**Keywords:** concrete construction; concrete durability; corrosion; deterioration; fatigue (materials); marine atmospheres; mix proportioning; permeability; reinforced concrete; reinforcing steels

## 2 Mehta

P. K. Mehta, F.A.C.I., is Professor in the Civil Engineering Department at the University of California at Berkeley. He is the author or coauthor of more than 120 technical papers and a book on microstructure and properties of concrete. In March 1988, he received the Wason Medal for Materials Research awarded by the ACI.

### INTRODUCTION

At the last CANMET/ACI Conference on the Performance of Concrete in Marine Environment, the author (1) reviewed the case histories of several concrete structures, which had shown damage on long-term exposure to seawater. From mineralogical analyses of deteriorated concrete samples, it was possible to describe the physical and chemical interactions between seawater and the typical constituents of portland cement paste. It was concluded that magnesium, sulfate, and chloride ions, and dissolved  $\text{CO}_2$  present in seawater, can indeed act as agents of destruction for all the hydration products of portland cement (namely, the calcium silicate hydrates, calcium aluminate and sulfoaluminate hydrates, and calcium hydroxide). The case histories clearly showed that although portland cement paste contained seeds of potential deterioration, these seeds did not germinate and grow into products which caused serious deterioration unless seawater was able to penetrate into the interior of concrete. Permeability rather than the chemistry of concrete was thus identified as the most important factor in long-term durability.

Based on a review of some recently reported case studies of concrete deterioration in marine environment, this paper takes a fresh look on the conclusions drawn by the author eight years ago. Also included here are recent developments, and a review of some current design codes as well as field practice for the construction of modern sea structures.

### RECENT CASE HISTORIES OF CONCRETE DETERIORATION

In 1979 the western half of the Hood Canal (Washington State) floating bridge broke up and sank during a heavy storm (2). Although corrosion of the reinforcing steel was not the principal cause of the mishap, inspection of the eastern half of the bridge revealed severe steel corrosion in several areas including the concrete deck slabs. It was suspected that the concrete had become permeable to seawater as a result of microcracks formed by numerous wetting/drying, heating/cooling, and loading/unloading cycles. Parallel to visible cracks, the corrosion in the reinforcing steel was found to be most severe (Fig. 1).

In 1980, spandrel beams of the San Mateo-Hayward bridge in California had to be repaired due to serious cracking of concrete caused by corrosion of the embedded steel (Fig. 2). High quality concrete (370  $\text{kg/m}^3$  cement content and 0.45 water-cement ratio) had been used for making both steam-cured precast beams and naturally-cured cast-in-place beams, which were made 17 years ago. No

cracks and corrosion were in evidence in the cast-in-place concrete, however all steam-cured beams showed cracks and corrosion at the underside and windward faces which were directly exposed to seawater spray. It is suspected that in the massive beams (7.9 x 3.7 x 1.8 m), a combination of heavy reinforcement and differential cooling rates in different parts of the beam (subsequent to the heating cycle in steam-curing) produced invisible microcracks, which later became continuous in the part of the structure that was exposed to severe weathering. Thus under adverse service conditions presented by the typical marine environment, even an impermeable concrete, could eventually become permeable and therefore vulnerable to a corrosion-cracking cycle, leading to serious structural damage.

With reinforced concrete structures exposed to marine environment, structural cracking does not lead to corrosion as long as the remaining concrete is impermeable, and especially if the concrete is fully submerged under water. It is assumed that reinforced concrete will crack in service, however the designers tend to limit the crack widths in the belief that a relationship exists between crack width and corrosion. Beeby (3) proposed that although larger cracks are likely to cause the breakdown of the passivity of the reinforcement rather earlier and hence initiate the corrosion earlier, there is no reason to expect the rate of corrosion to be influenced by the surface cracks unless these cracks represent widespread internal cracking.

Many investigations are reported in the published literature on the rates of diffusion of oxygen, water vapor, and chloride ions in hydrated cement pastes because their presence at the surface of steel is necessary before galvanic corrosion cells can form. In high quality concrete the transport of gases and ions by diffusion is an exceedingly slow process, however it is relatively faster by permeation through an interconnected network of microcracks and capillary voids. In a diagrammatic presentation (Fig. 3), Mehta and Gerwick (2) proposed that such a network of microcracks already exists in concrete and that the widening of microcracks in service due to numerous causes would precede the corrosion process.

Researchers with practical experience in concrete durability problems seem to concur with the viewpoint expressed by Mehta and Gerwick (2). For example, Idorn\* says, "I agree fully that cracking in concrete is the primary precursor to corrosion, also where chloride ions are involved. By nature reinforced concrete has cracks, and due to practical conditions most structures have cracks from the beginning that are wide enough to cause ingress of saline water by capillary action. Naturally in this way as soon as chloride ions reach the rebar, one has a powerful anodic-cathodic potential, and with free moisture abundantly present, corrosion will proceed. The possible concurrent ingress of chloride ions by diffusion will be a much slower process."

Gjorv and Kashino (4) were able to obtain valuable data from a detailed investigation on durability of concrete in a 60-year old reinforced concrete pier in Oslo harbor, before the structure was demolished. Originally, a non air-entrained concrete of 25 to 30 MPa quality (350 kg/m<sup>3</sup> cement content, 0.53 water-cement ratio) had been prescribed. However after 60 years of seawater exposure the concrete in the deck had a compressive strength of 42 - 45 MPa and

\*G. M. Idorn, private communication, October 8, 1986.

## 4 Mehta

a permeability coefficient of  $2.5$  to  $8 \times 10^{-13}$  kg/Pa m.s. The permeability of concrete in the pillars was even lower. Although an ordinary portland cement with a relatively high  $C_3A$  content ( $> 9\%$ ) had been used, the low permeability of the concrete provided satisfactory chemical resistance. This confirms the observation made by several other investigators cited earlier by the author (1) that, with concrete exposed to marine environment the high  $C_3A$  content of the cement has no adverse effect on durability as long as the concrete has a low permeability. Only to a depth of approximately 5 mm and in large air voids, precipitation of ettringite crystals was noted by Gjorv and Kashino (4). The authors reported the existence of some microcracks to a depth of about 10 to 35 mm, but these cracks were suspected to be the result of thermal stresses during construction.

By subjecting concrete samples to freezing and thawing tests in the laboratory, it was confirmed that the concrete had a poor frost resistance. In the tidal zone, all pillars had either been repaired or were severely damaged by the action of frost. An analysis of the air-void system of a concrete sample showed 1.7% total air content, of which voids below 300  $\mu\text{m}$  comprised only 0.4%, and the average void spacing was 0.35 mm.

Gjorv and Kashino (4) found very interesting results in regard to depth of carbonation, chloride penetration, and corrosion of steel in concrete. In the upper part of the deck and in the fully submerged part, the carbonation depth was quite low (1 to 8 mm); only in the lower part of the deck carbonation depths up to 33 mm were recorded. Similarly, in the upper part of the tremie-cast pillars the depth of carbonation was high (23 to 24 mm) but in the tidal zone and submerged parts it was quite low (1 to 7 mm). In the lower part of the deck, the chlorides had penetrated to a depth of more than 80 mm and the chloride content at the steel bar was 0.05 to 0.1% by weight of concrete. In the fully submerged part, the corresponding chloride content was 0.3 to 0.35%.

The pier had been subjected to frequent maintenance and repair jobs during 60 years of service life. The first repairs were carried out after 10 years because of steel corrosion in deck beams; however, due to deterioration of concrete from frost action extensive repairs (by recasting and jacketing) were carried out after 16 years of service. The authors observed that cleaning and recasting of cracks in spalled areas from steel corrosion did not stop subsequent corrosion because after a few years new cracks appeared in the adjacent areas.

Although deck beams at the inner and most protected parts of the pier were in good condition, at the outer and exposed part of the pier all deck beams had suffered cracking due to rebar corrosion. Many slab sections showed a similar problem. Several of the pillars also cracked from rebar corrosion in the top part above the tidal zone; the fully submerged parts of the pillars did not show any rust staining or corrosion cracking. The findings of Gjorv and Kashino (4) are generally in accord with this author's conclusions (1), which were based on a review of case histories of other structures that had similarly been exposed for a long time to seawater.

Recently, Gerwick\* found a case of very extensive corrosion of reinforcing steel on a major seawater transport project in the Middle East. He believes that the corrosion of steel due to high chloride permeability was made possible because of multiple microcracks in concrete, which occurred as a result of frequent thermal and humidity changes (extremely hot days followed by cold nights). The use of porous and permeable limestone as a coarse aggregate in the concrete probably contributed to the problem.

### CONCLUSIONS FROM THE CASE STUDIES

From a review of case studies described above and in the earlier publication (1), the author firmly believes that the permeability of concrete is indeed the key to overall durability. For construction of durable concrete sea structures, the following lessons from the past should be helpful:

1. Harmful chemical reactions between the constituents of hydrated cement paste and seawater, such as carbonation, sulfate attack, and magnesium ion attack, can be limited to the surface when well known measures to assure low permeability of concrete have been rigorously implemented.

Unreinforced concrete structures made with low water-cement ratio and high cement contents (e.g., more than  $350 \text{ kg/m}^3$ ) have generally shown a satisfactory service life in marine environment except when they had been exposed to high thermal gradients, frost action, or alkali-aggregate attack. The structures built since 1950's have usually been protected from frost action by proper air entrainment, and from alkali-aggregate attack by the use of low-alkali portland cement and/or pozzolanic additives. Both carbonation attack and sulfate attack will essentially be negligible in modern sea structures made with low-permeability concrete mixtures.

2. The causes of high permeability in newly built concrete structures are not limited to poorly proportioned concrete mixtures, such as those with low cement content, high water-cement ratio, and improper aggregate grading. With well-proportioned concrete mixtures a frequent cause of high permeability is the poor concreting practice, such as incomplete mixing of concrete, inadequate consolidation and curing after placement, insufficient concrete cover over reinforcing steel, and badly constructed joints. Areas of heterogeneity in the microstructure of hardened concrete are potential sites for development of microcracks.
3. Even well-constructed structures, made with high-quality concrete mixtures, can become permeable in service due to excessive cracking caused by various reasons, such as settlement, premature loading, overloads, and repeated impact. This is especially the case with reinforced concrete, which contain microcracks to begin with, as discussed next.

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\*B. C. Gerwick, private communication, 27 February 1987.

## 6 Mehta

4. With reinforced concrete structures, it is a common observation that concrete spalling and cracking are frequently accompanied by corrosion of the embedded steel. The reinforcement does not prevent the cracking of concrete due to drying or thermal shrinkage; by restraining the shrinkage of concrete the steel reduces the crack widths so that instead of a few wide cracks numerous microcracks are formed. It is the widening of these cracks in service due to various causes that facilitates the permeation of ions and gases, which are needed for the corrosion process to proceed. Corrosion leads to further cracking, and this cracking-corrosion cycle results in enhanced permeability and eventually in serious structural damage.

In short, to obtain long-term durability of concrete sea structures, the control of concrete cracking in service is of as much importance as the control of concrete permeability through proper mix proportioning and concreting practice.

### RECOMMENDED PRACTICE FOR MODERN CONCRETE SEA STRUCTURES

Lessons from case studies of concrete deterioration by seawater, as summarized above, have been generally incorporated into industry codes which, when carefully followed, should enable us to build concrete sea structures that will remain serviceable for longer periods than the structures built in the past. However, many modern sea structures built during the last 10 to 15 years are required to withstand unprecedented demanding stress conditions. For example, offshore concrete structures in the Arctic and the North Sea, which are exposed to enormous hydrostatic and differential pressures, not only require very high strength but also resistance to abrasion and impact from floating ice, and are heavily reinforced as well as prestressed.

To protect the embedded steel from corrosion, it has become imperative that concrete mixtures with very low permeability are used. The recommended practice for durability of reinforced and prestressed concrete in offshore structures has for obvious reasons the most stringent requirements in the construction industry. A brief review of the recommended practice will be useful here because the technology of making high strength-high impermeability concrete has generated considerable research and development in the field of new materials and concreting practices.

In the United States and Canada, the report by ACI Committee 357 (ACI 357R-84) is used for the design of fixed reinforced and prestressed concrete structures for service in a marine environment. In Europe and Asia, for the design and construction of concrete sea structures it is customary to use the recommendations of the International Federation of Prestressed Concrete Structures (FIP, 4th edition, 1984). From standpoint of durability both the ACI and FIP codes provide that a coastal or offshore structure be divided into three zones: the SUBMERGED ZONE where the concrete is continuously under water, the SPLASH ZONE where the concrete is constantly subjected to wetting and drying, and the ATMOSPHERIC ZONE which is above the splash zone. Since the severity and type of durability problems are different for different zone, the properties of a concrete mixture need not be the same for the entire structure.

Generally speaking, for long-time durability of coastal and offshore structures the concrete must show resistance to: THERMAL CRACKING, FROST ACTION, ABRASION/EROSION LOSS, and EXPANSIVE CHEMICAL AND ELECTROCHEMICAL PHENOMENA (such as alkali-aggregate reaction and corrosion of reinforcing steel). The permeability of concrete is important in all three zones because it influences all physical and chemical phenomena causing concrete deterioration. It should be noted that the abrasion/erosion loss is usually limited to the splash zone whereas most other durability problems can occur both in the splash zone and the atmospheric zone (1).

On long-time exposure to seawater the concrete in the submerged zone, if permeable, may experience a mild deterioration, caused by chemical attack. The rate of attack is very slow because the insoluble products of chemical interaction between seawater and hydrated portland cement paste - such as brucite (magnesium hydroxide), aragonite (calcium carbonate), and ettringite (calcium sulfoaluminate hydrate) - deposit in voids and microcracks, thereby reducing the permeability of concrete. From a study of the pore size distribution of hydrated portland cement paste by mercury intrusion porosimetry, Midgley and Illston (5) have concluded that even the penetrating chlorides reduce the size of small pores thus reducing the permeability. It should be noted that this sort of protection from the sealing of voids and microcracks by the formation of insoluble products will not be available in the splashing zone because there the structure is constantly under the influence of tidal action and impacting solids, such as sand, gravel, and ice. As a result of loss of cover due to abrasion and microcracking, the concrete gradually become more permeable and, therefore, vulnerable to other types of attacks.

Although both the ACI and FIP recommendations for design and construction of concrete sea structures were published quite recently (1985), it is not surprising that the recommended practice lags behind the current (1986-87) field practice. A sufficient amount of published data is now available from field experience in the North Sea where 20 offshore concrete platforms have been constructed during the period 1972-87 (6,7). It will be useful therefore to compare the North Sea field practice with the ACI and FIP recommendations regarding the selection of concrete-making materials, concrete mix proportions, and construction practice for long-time durability. The recommendations on the selection of materials and mix proportions are summarized in Tables 1 and 2, respectively, and will be discussed next.

#### Selection of Concrete-Making Materials

Cement Composition--The ACI maximum limit of 10%  $C_3A$  in portland cement is based on the questionable assumption that cements with higher than 10%  $C_3A$  are susceptible to sulfate attack in seawater. Although seawater typically contains 2700 mg/liter sulfate ions, which is nearly twice the concentration for which the ASTM Type V portland cement (< 5%  $C_3A$ ) is generally recommended, the author (1) cited three case studies where no sulfate attack was observed in long-time (46 to 67 years) seawater exposure of concretes made with cements containing 14 to 17%  $C_3A$ . These were low-permeability concretes with high cement content. Similarly, Mather (8) confirmed that concrete prisms made with a high- $C_3A$  (12.5%) ASTM Type III portland cement, did not show any lack of chemical durability after more than 30 years of seawater exposure at Treat Island, Maine. It seems that ettringite does form as a result of chemical interaction

## 8 Mehta

between seawater and portland cement paste but the formation of ettringite in the presence of seawater is not accompanied by expansion and cracking. A possible explanation for this phenomenon is that ettringite expands only when it forms in an environment saturated with hydroxyl ions. The deleterious expansion does not occur when hydroxyl ions in a cement paste have been replaced with chloride ions (9).

From standpoint of chloride corrosion of steel, recent research studies seem to provide some justification for a minimum  $C_3A$  limit in portland cement, although it is worth emphasizing that the chloride diffusion alone is not sufficient to cause corrosion. Page et al (10) investigated the influence of different cement types on rates of chloride diffusivity in the hydrated cement pastes. The diffusion rates for chloride ions in ASTM Type I cement pastes were not found to be significantly different between cements with 7.7%  $C_3A$  and 14.3%  $C_3A$ . On the other hand, an ASTM Type V portland cement (1.9%  $C_3A$ ) showed a significantly higher chloride diffusion rate than Type I portland cements. Recent work by Rasheeduzzafar et al (11), from laboratory and field tests in the Middle East, has confirmed that the chloride diffusion rate with Type V portland cement paste is somewhat higher than with Type I portland cement. More importantly, blended portland cements containing 30% Class F fly ash or 65% granulated blast-furnace slag were found to produce pastes with considerably lower diffusion rates than portland cements.

As for the current field practice, it may be noted from Table 1 that for the construction of Condeep SP Gullfaks C the Norwegian Contractors switched from a coarser Type I portland cement to a finer Type II (moderate  $C_3A$ ) portland cement. This was done from cement-superplasticizer compatibility considerations and to obtain better slip-forming characteristic in concrete.

It seems that the limits on the  $C_3A$  content of cement, such as given by the ACI Recommended Practice, are outdated. The fineness and the  $C_3A$  content of a cement may be important from the standpoint of their influence on the rheology of superplasticized concretes, and not due to any long-term durability effects as implied in the ACI recommendations. Theoretically, for concrete sea structures the use of any ordinary portland cement (6-12%  $C_3A$ ) should be satisfactory provided the cement is compatible with the admixtures to be used which, of course, among other things, will also depend on the alkali content of the cement. In the current field practice heat of hydration and permeability considerations require that, respectively, the use of the ASTM Type III and Type V portland cements be discouraged and the use of portland-pozzolan cements and portland-blastfurnace slag cements be given special consideration.

Admixtures--There seems to be sufficient evidence that, for obtaining impermeability and abrasion resistance\* which are essential for long-time durability of concrete exposed to seawater, both water-reducing and mineral admixtures should be used. The FIP recommendation provide that high-quality pozzolanic materials, such as special silica fumes may be added to produce improved strength, durability, and workability. However, contrary to overwhelming evidence from both laboratory and field practice, the ACI 357R-84 does not appear to take a clearly supportive stand in favor of the use of

\*Abrasion/erosion resistance is needed mainly in the splash zone.

pozzolanic and cementitious admixtures in concrete sea structures (silica fume and ground granulated blast-furnace slag are not even mentioned in the ACI recommendation). However, some recent publications provide a wealth of useful information on the composition and properties of important concrete admixtures (12-16).

This author believes that by improving the workability, reducing the heat of hydration, and increasing the strength of the transition zone between aggregate and cement paste, fine particles of a mineral admixture are able to enhance the homogeneity in hardened concrete microstructure. This improves the ability of a concrete to resist microcracking, which is important for maintaining the impermeability during service. The amount of a mineral admixture needed for this purpose will vary with its particle size and chemical composition. Whereas 15 to 25% fly ash by weight of total cementitious materials may be adequate, generally 50 to 70% granulated blast-furnace slag is needed to achieve similar properties. Due to the exceedingly small particle size, only 5 to 10% condensed silica fume is usually enough.

Similarly, to enhance the homogeneity in hardened concrete microstructure, it is essential to use a low water-cement ratio and to obtain a proper dispersion of the cementitious materials in a fresh concrete mixture. Water-reducing admixtures are commonly used for this purpose. In earlier work (Condeep Beryl A, 1974), a lignosulfonate type water-reducing agent was employed. This type of admixture is produced from an industrial by-product containing certain impurities which, when present in excessive amounts, are known to cause considerable air entrainment and set retardation in fresh concrete mixtures. High-purity naphthalene or melamine sulfonate type water-reducing admixtures, also called superplasticizers, are now preferred because they do not lead to excessive retardation and air entrainment. Furthermore, as illustrated by Fig. 4, the naphthalene or melamine sulfonates are better dispersing agents for cement-water suspensions than lignosulfonates. This is because lignosulfonate molecules exhibit a high degree of cross-linkage and form spherical microgel flocs which, unlike the linear molecules of a superplasticizer, are not able to cover the cement grains efficiently.

Air entraining admixtures are now invariably used in concrete mixtures when resistance to cracking due to freezing and thawing cycles is desired. With ordinary concrete (0.4 or higher water-cement ratio), air entrainment with a minimum spacing factor of 0.2 mm is recommended. Researchers including Malhotra (17) have shown that it is difficult to entrain sufficient air with the desired minimum spacing factor when the silica fume content is more than 20-30% by weight of cement. In laboratory tests by the author, air-entrained concretes with up to 15% silica fume performed satisfactorily to freezing and thawing cycles (ASTM C 666 Procedures A and B). Pigeon et al (18) reported from laboratory tests on concretes containing 9% condensed silica fume and 0.3 water/cement ratio that enough freezable water was present in these low water cement ratio specimens to cause damage in freezing and thawing tests, unless air entrainment was used. With the particular materials and test conditions used by the authors, the value of critical spacing factor was 400  $\mu\text{m}$  for the silica fume concrete, and 400  $\mu\text{m}$  without the silica fume. As a safeguard against the loss of air on incorporation of a fine sand, a pozzolan, or a superplasticizer in a concrete mixture, it is essential that the air content and the void spacing are confirmed by taking core samples of hardened concrete from the structure.

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Both ACI and FIP warn that when two or more admixtures are used, their compatibility with the cement and aggregate types to be used for concrete-making, should be documented. Also, to protect reinforcing and prestressing steel from corrosion, no  $\text{CaCl}_2$  or admixtures containing chlorides should be used.

In regard to the North Sea field practice, it may be noted that the concrete for Gullfaks C (1986-87) contains 5% condensed silica fume by weight of cement and 6 liters/m<sup>3</sup> of a naphthalene sulfonate superplasticizer (7). Air entrainment was used for the splash-zone concrete, and a spacing factor of 250  $\mu\text{m}$  was obtained with 3 to 5% air.

### Aggregates

ACI 357R permits any natural sand and gravel or crushed rock that conform to ASTM C 33 specifications for concrete aggregates. FIP recommends that aggregates likely to undergo physical or chemical changes should be avoided. In addition to these recommendations there are other points discussed below, which must be considered in the selection of aggregate for obtaining long-time concrete durability.

Microcracking in the transition zone between coarse aggregate particle and cement paste seems to be the most important cause for the increase of concrete permeability in service. The particle size, shape, and mineralogy has a great influence on the strength and therefore the microcracking behavior of the transition zone. Generally, normal-weight aggregate particles that are larger than 20 mm and either too elongated or too flat tend to accumulate bleed-water in their proximity. This results in a porous and weak transition zone, which microcracks easily and can thus become a contributing factor in increasing the permeability of concrete. Table 1 shows that the maximum size of aggregate for concrete used in the construction of Condeep structures in the North Sea has been reduced from 32 to 20 mm, although the recommended practice in the codes makes no attempt to suggest a relationship between the maximum size of aggregate and the permeability of concrete.

Sandstones and quartz aggregates do not form strong chemical bonds with portland cement paste and their coefficients of thermal expansion are also considerably higher than the cement paste. On the other hand, limestone and lightweight aggregates not only form strong chemical bonds but also show less mismatch with the cement paste matrix regarding thermal and elastic properties. Consequently, in seawater exposure, concrete mixtures made with limestone aggregate (2), and with lightweight aggregate (19) have shown better durability than similar concrete mixtures made with quartz or sandstone aggregates.

The aggregates for the splash zone concrete, where resistance to heavy erosion is required, should be hard and strong. Also, a relatively high coarse to fine aggregate ratio (e.g., 65 to 35%) should be used. Field experience with moderately good-quality concrete (30 to 40 MPa) and also with high-quality concrete (50 to 60 MPa) has shown that under conditions of severe abrasion/erosion, it is usually the coarse aggregate-cement paste bond that fails, which causes the aggregate particles to pull out. The incorporation of a highly reactive pozzolanic material, such as condensed silica fume into a concrete mixture improves considerably the aggregate-cement paste bond and, consequently, the

abrasion/erosion resistance of the concrete. Gjorv et al (20) reported that a very high-strength concrete mixture (150 MPa), containing a jasper or quartz-diorite aggregate, a superplasticizer, and condensed silica fume, showed an abrasion or wet-erosion loss of the same order as that of a granite element used for reference purposes.

### Concrete Mixture Specifications

Specifications for structural concrete usually contain a minimum 28-day compressive strength requirement, a maximum permissible water to cement ratio, a minimum cement content, and a minimum consistency for placement and consolidation of the fresh concrete mixture. Except freeze-thaw resistance, the 28-day strength is generally used as an index of concrete permeability and durability. In Table 2 the ACI and FIP recommendations on concrete mix proportions for offshore structures are compared with the North Sea field practice. In spite of the severity of seawater environment, it is regretful that test methods and specifications for concrete permeability have not yet found their way into the ACI and FIP recommended practice.

The compressive strength requirements in Table 2 show that both the FIP and the ACI strength specifications are behind the North Sea concrete construction practice by approximately 15 years. The Ekofisk and Beryl A Condeep structures, built in early 1970's, contained concrete that barely qualifies as high-strength (40 MPa). The modern concrete sea structures in the North Sea are built with very-high-strength (minimum 60 MPa) concrete (7). For instance, the specified strength (56 MPa) for Gullfaks C concrete is approximately 50% higher than the Beryl A concrete (36 MPa). The actual 28-day compressive strength of the Gullfaks C concrete core samples was found to be approx. 70 MPa. It should be noted that, in general, the higher the strength the higher will be the impermeability.

Both the FIP and the ACI recommendations require a minimum cement content of approx.  $360 \text{ kg/m}^3$  ( $600 \text{ lb/yd}^3$ ), however the former recommends a minimum of  $400 \text{ kg/m}^3$  cement for the splash zone concrete mixture. The use of high cement content in massive structures frequently leads to thermal cracking, which has the effect of increasing the permeability and reducing durability. In 1970's some thermal cracking was noticed in an offshore concrete structure. Consequently for the construction of Statfjord C (1981), the cement content was limited to  $380 \text{ kg/m}^3$ . In spite of a lower cement content, approximately 20% higher strength than the Beryl A concrete ( $430 \text{ kg/m}^3$  cement) was obtained by the use of a finer cement, a smaller size aggregate, and a superplasticizer.

Compared to the Beryl A concrete, with Gullfaks C (1986) more than 50% increase in the concrete strength was achieved by using essentially the Statfjord C concrete mixture except that the cement content was increased from 380 to  $430 \text{ kg/m}^3$  and, additionally, 5% of a reactive pozzolan ( $20 \text{ kg/m}^3$  silica fume) was used as a supplementary cementing material. In order to prevent thermal cracking adequate measures were taken to control the temperature gradients in concrete.

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## CONCRETE PRODUCTION AND CONSTRUCTION PRACTICE

The realization that proper selection of materials and mix proportioning alone will not produce a durable concrete structure has made the construction industry pay more attention than before to the **neonatal history of concrete** (early age from birth). In an earlier publication,\* the author said:

In the medical profession it is well recognized that in order to develop into a healthy person, a newborn baby needs special attention during the early period of growth. Something similar applies to concrete. Concrete technologists agree that deficiencies acquired by freshly made concrete due to loss of workability at or before the placement, segregation and bleeding during the consolidation, and abnormal rates of setting and hardening, can impair a concrete permanently and reduce its service life.

It is not possible here to review in detail the recommended concrete production and construction practices. However, from standpoint of durability of concrete, summarized below are some of the useful guidelines that have emerged from construction experience with modern concrete structures.

Batching--The batching sequence and methods should be aimed at producing a concrete mixture, which is as homogeneous as possible. For instance, the deflocculation of the cement particles as well as particles of other fine materials, such as condensed silica fume, with a plasticizing admixture may be carried out by high-speed mixing before the aggregates are added. If the fine aggregate is dry, a part of the mixing water may be used to moisten it before adding the material to the concrete mixer.

Moksnes et al (7) found that stable air was not easily entrained in high-slump concrete containing a naphthalene sulfonate superplasticizer. The problem was solved by a modified batching procedure, which involved two-stage batching so that the air was first entrained in a stiff-consistency concrete mixture.

Mixing and Transporting--Properly mixed concrete may be conveyed by any suitable means to the placement site, however it is important that the **homogeneity** of the fresh mixture is not adversely affected during the transportation. According to Gerwick (6), with ready-mixed trucks a continuous agitation of the **entire batch** must be assured. This means that the blades must be frequently checked for excessive wear so that there is no significant buildup of segregated concrete inside the mixture.

Placement--When placed by pumping and pressures are excessive, a part of the mixing water may be lost due to absorption into the aggregate. This may lead to slump loss and the associated problems, such as poor consolidation and finishing. Some entrained air content may also be lost during pumping. Although, with 20 mm maximum aggregate size the ACI Building Code 318 requires 6% minimum air content in severe freeze-thaw exposure, Moksnes et al (7) report that 3 to 5% actual air with a spacing factor of 0.25 mm (in core samples of

\*Reference 9, p. 301.

hardened concrete), is adequate for durability of low water- cement ratio concrete mixtures containing silica fume and superplastizer.

Consolidation--Following the placement, proper consolidation is necessary to achieve a uniform distribution of all the solids, water, and air present in the concrete mixture. Application of the vibrator to freshly placed concrete causes a breakdown in mechanical stacking, reduces trapped air voids, and brings any excess water to the surface where it can be lost by evaporation.

When "flowing concrete" containing superplasticizers were first introduced, it was erroneously claimed that no consolidation would be necessary. However, superplasticized concrete mixtures of even high slump (200 to 250 mm) tend to be thixotropic and, unless vibrated, they will not flow properly when placed in areas heavily congested with reinforcing steel. Forssblad (21) reports that, without consolidation, reinforced concrete shows a tendency to entrap 3 to 5% air, a reduction in bond strength, and settlement cracks around the reinforcement. In other words, even superplasticized concrete should be vibrated. However, according to the author (21), with superplasticized concrete the radius of action of an internal vibrator is 20 to 30% greater than with ordinary concrete of plastic consistency and therefore the vibration time can be reduced.

Since imperfections in concrete are undesirable, it is preferable to overvibrate than to underconsolidate. According to Whiting et al (22) overconsolidation to the point of incipient segregation results in improved properties, such as higher compressive strength and lower permeability to chloride ions.

Finally, to assure a low permeability in concrete mixtures which bleed on consolidation, some recommend revibration. Particularly in high water-cement ratio concrete mixtures it has been observed that when water departs the concrete mixture during the bleeding process, it leaves behind microcapillaries. Revibration provides an opportunity to break up these microcapillaries. According to Welton (23):

During the drying process, the water works its way out through a system of capillaries. If the finished concrete has a neatly defined river and tributary system of capillaries running through it, the concrete has a built-in system ready for its own destruction. Tearing up the capillaries by diligent, dynamic vibrator work is the last chance before curing to achieve quality. Need we ask further, Why (re)vibrate?"

A recently issued report of the ACI Committee 309 (24) contains useful information on many aspects of concrete consolidation including imperfections, undervibration, overvibration, and revibration.

Surface Finishing--Even with massive structures, a smooth finish free from surface voids and defects is very important to reduce permeability of the surface layer which, in fact, serves as the first line of defense against the ingress of water and aggressive fluids.

Sadegzadeh et al (25) reported that the abrasion resistance of concrete was greatly influenced by the pore structure of the cement matrix in the surface zone (outermost 2 to 3 mm). The pore structure was influenced by differences in water-cement ratio and surface finishing procedures, which ultimately affected the

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abrasion resistance of concrete. For instance, compared to manual finishing and troweling, power finishing operations produced a harder and more abrasion-resistant surface. Repeated power trowelling, after the moisture brought out by the previous trowelling operation had been allowed to evaporate, gave the best results.

Concrete mixtures with high cement content, and especially those containing condensed silica fume, tend to be sticky and difficult to finish. In such cases special vibratory screeds should be used for obtaining a smooth and impermeable surface.

Curing--The maintenance of proper humidity and temperature conditions for a certain minimum time period after placement is as essential for obtaining the desired impermeability in concrete as all other factors discussed before. For instance, in this regard the FIP recommendations require that in cold weather the concrete at the time of placing should be at least at 5°C and should be maintained above this temperature until it has reached a minimum strength of 5 MPa. The temperature may be raised by heating the mixing water and/or the aggregates, however the cement should not be allowed to come into contact with water at a temperature higher than 60°C. Also, to ensure maximum durability the concrete surface should be kept covered with fresh water. Seawater should not be used for initial curing of reinforced or prestressed concrete. In the current Norwegian practice, it is permitted to submerge high strength concrete members in seawater after three days of normal curing because the concrete is assumed to have become virtually impermeable by this time.

When there is doubt about the ability to keep concrete surface permanently wet for the whole curing period, or where there is a danger of thermal cracking due to too cold a curing water, a heavy-duty membrane curing compound should be used to seal the concrete surface. According to Gerwick (6) since most concrete mixtures used for sophisticated offshore structures are essentially impermeable, the emphasis today is primarily on sealing the surface against loss of moisture rather than on supplying the curing water externally. He warns, however, that with membrane curing compounds, heat from the sun or cement hydration may degrade the curing compound, and therefore one or more additional applications may be necessary during the first day.

Concrete Cover--For offshore concrete sea structures, with 0.5 m or more wall thickness, the nominal concrete cover for protection of reinforcing steel and prestressing tendons is related to the zone of exposure. In this regard, both FIP and ACI 357R have similar requirements. In the submerged zone, 50 mm cover thickness over principal reinforcing steel and 75 mm over prestressing tendons are recommended. In the splash zone and the atmospheric zone, which are subject to seawater spray, the recommended cover thickness is 65 mm for reinforcing steel and 90 mm for prestressing steel. Stirrups may have 13 mm less cover than the preceding recommended values.

When the concrete cover cracks due to any reason, it is observed that the crack width is directly proportional to the cover depth. Therefore, Gerwick\*

\*B. C. Gerwick, personal communication.

feels that with modern impermeable concrete mixtures, it is not necessary to have more than 50 mm cover thickness for bolt reinforced steel and prestressing steel, provided the cover is truly dense. Due to the problem of cracking-corrosion interaction as discussed above, ACI 224R-80 recommends 0.15 mm maximum crack width at the tensile face of reinforced concrete structures subject to wetting and drying or seawater spray. FIP recommends that crack width at points nearest to the main reinforcement should not exceed 0.2 mm for 50 mm, or 0.3 for 75 mm depth of cover.

From field tests on durability of steel fiber-reinforced concrete exposed to marine environment, Mangat and Guruswamy (26) reported that chloride concentrations in the vicinity of cracks increased with increasing crack width, the effect being significant above 0.5 mm crack width and insignificant at 0.2 mm or less.

Construction Joints and Cold Joints--In reinforced and prestressed concrete structures, many durability problems originate from water seepage through construction joints and cold joints. To obtain watertight construction joints the surface should be carefully prepared by sand blasting or a high-pressure water jet until all dirt or laitence is removed and the coarse aggregate is exposed to a depth of about 6 mm. Cold joints may be prevented by the use of set-retarding admixtures, or specially formulated mortar mixtures containing styrene-butadiene latex.

Control of Thermal Cracking--Thermal cracking will probably be the principal factor contributing to the permeability of a high-strength concrete, when measures to protect concrete from steep thermal-stress gradients are not taken. According to FIP recommendation, when the minimum dimension of concrete to be placed at one time is greater than 600 mm and the cement content is more than 400 kg/m<sup>3</sup>, the use of a cement with a slower release of heat of hydration and other methods to reduce the temperature rise should be considered. Modern methods of controlling the temperature rise include using mineral admixtures to replace a portion of the portland cement in concrete, reducing the rate of concrete placement, precooling aggregates, using ice to replace a part or all of the mixing water, and cooling the fresh concrete mixture with liquid nitrogen.

In the Norwegian offshore concrete practice (7), for large and insulated pours the typical temperature rise can be on the order of 12 to 14°C per 100 kg cement (approx. 50°C total temperature rise). Reduction of cement content (partial substitution of cement with a pozzolan), addition of crushed ice, and the use of cold water to cool fresh concrete are among the measures proposed to control thermal cracking. It is also helpful to reduce the rate of temperature drop from a concrete structure by insulating the formwork and the surface of concrete.

#### OTHER CAUSES OF CRACKING IN SERVICE

Generally, the first cracks in concrete sea structures may form due to one or more of the following causes: **improper loading** (viz., premature loads, overloads, settlement, or high impact such as in the case of pile driving), **thermal cracking** (i.e., rapid cooling of the exterior when the temperature in the interior of concrete is still high due to uncontrolled rise in temperature from heat of

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hydration), and **frost action** (i.e., exposure of concrete to repeated freezing-thawing cycles when the concrete does not contain enough entrained air with proper void spacing. On long time exposure to cyclic loads, **fatigue cracks** may also appear in the form of multiple microcracks, extending all the way to rebars. In reinforced and prestressed concrete structures once the permeability of concrete has increased, further cracking will occur from the **corrosion of steel**. The causes and control of cracking due to thermal gradients and frost action in high-strength concrete have already been discussed; a brief review of cracking from fatigue and from corrosion of steel in concrete, will now be presented.

#### Fatigue Behavior of High-Strength Concrete

At the Conference on Utilization of High Strength Concrete, several papers were presented on the subject of fatigue of concrete (25-28). Lenschow (27) expressed the concern that changing from normal-strength concrete to high-strength concrete will lead to reduced section thickness and higher dead loads, thus causing a heavier fatigue loading. From limited test data the author reported that the fatigue behavior of high-strength concrete seems to be similar to that of normal-strength concrete.

Nelson et al (28) investigated the fatigue behavior of plain high-strength concrete subjected to biaxial-cyclic compression. Results indicated that with specimens tested in-fatigue at maximum stress levels below approx. 76%, failure occurred at a higher number of load cycles as the biaxial stress ratio decreased. The endurance limits for high-strength concrete for a fatigue life of 2 million cycles were found in the range of 47 to 52% of the uniaxial-static compressive strength.

Although most of the research on fatigue of concrete is devoted to compressive stress, it is recognized that tensile fatigue cracking is a more important factor in the design of bridges and pavements exposed to seawater spray. Lambote and Taerwe (29) investigated the fatigue of plain, normal-weight, high-strength concretes subjected to flexural tensile stresses. The authors did not find any significant difference between the SN-curves for high-strength and normal-strength concretes. Although high-strength concrete showed a slightly better fatigue life for the stress range  $0.7 < \sigma_{\max}/f_{cm} < 0.9$ , but the inverse could be true for lower values of  $\sigma_{\max}/f_{cm}$ .

For design of offshore structures, there is a considerable research interest in high-amplitude and low-cycle fatigue behavior of concrete at high-stress levels (70 to 90% of static compressive strength). Nishiyama et al (30) reported that under submerged condition some reduction took place in the fatigue life of both plain concrete under compression and prestressed concrete beams in flexural loading. The reduction in the compressive fatigue life was attributed to acceleration of microcrack propagation by the wedging action of adsorbed water, however with the flexural fatigue life the cause was suspected to be the pumping action of water trapped in the flexural cracks.

Also, with increasing demand for deep water and floating structures in the ocean there has developed some interest in the behavior of high-strength, lightweight concrete. A mobile drilling structure, made with 60 MPa lightweight concrete, is now in use in the Beaufort Sea, Alaska (31). According to Kepp and Roland (32), Norwegian Contractors have developed a 70 MPa lightweight concrete

(28-d compressive strength), containing 385 kg/m<sup>3</sup> cement, 15 kg/m<sup>3</sup> silica fume, and a plasticizing admixture (200 mm slump at a water/cement ratio of 0.42). Preliminary results from a recent study\* at the University of California (Fig. 5) provide evidence that for the same compressive strength the bond strength (pull-out test) and the fatigue strength of a reinforced superplasticized lightweight concrete containing silica fume, are considerably superior to the corresponding values for both the normal-weight concretes (with and without silica fume) and a lightweight concrete without silica fume. This is probably because of a better elastic match between the high-strength lightweight aggregate particles and the superplasticized cement paste matrix containing the silica fume (a strong transition zone).

### Corrosion of Steel in Concrete

The galvanic cells in the corrosion of steel in concrete consist of an anode process and a cathode process (Fig. 6). The anode process involving the ionization of metallic iron does not begin until the passive iron oxide film, normally present on the surface of steel, is either removed by an acid attack (e.g., carbonation of concrete) or is made permeable by the action of Cl<sup>-</sup> ions. A commonly accepted chloride threshold value for depassivation of steel is 0.2 percent total chloride ion by weight of cement (i.e. about 0.8 kg/m<sup>3</sup> for typical high-strength concrete). With modern sea structures, the low water-cement ratios used in concrete mixtures essentially eliminate the threat of carbonation to any significant depth. Therefore, the depassivation of the embedded steel is influenced mainly by the rate of chloride penetration, which depends on the permeability of concrete and depth of cover.

Once the passivity of steel is destroyed, it is the cathode process and the electrical resistivity of the system which control the rate of corrosion. The cathode process, which is necessary to complete the corrosion cell, will not progress until a sufficient supply of both oxygen and moisture is available at the surface of steel at some distance away from the anode. Again with modern sea structures the availability of oxygen at the steel surface will be severely restricted by the low permeability and the high cover thickness of concrete. The oxygen is generally nonexistent in the submerged zone due to negligible solubility in salt water.

Permeable (moist) concrete is a good conductor of electrons between the anode and the cathode. Hope et al (33) reported that the resistivity of concrete increased with increase in age of curing and decrease of water/cement ratio and the curing temperature, and that the probability of corrosion changed from very likely to very unlikely as the concrete resistivity increased from 6500 to 8500 ohm cm. in chloride contaminated specimens. With dense and impermeable modern concretes (especially those made with low water-cement ratio, high cement content, and pozzolanic and water-reducing admixtures) the electrical resistivity is generally higher than 8500 ohm cm, and therefore no corrosion should occur. In such concretes, the moisture and chlorides must penetrate from outside to lower the electrical resistivity before any significant corrosion can occur.

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\*Ph.D. dissertation of A. Mor, 1988 (supervisor, Professor B. C. Gerwick).



In conclusion, the occasional failure of reinforced concrete structures due to corrosion of the reinforcing steel can only have occurred **when all three protection mechanisms** namely, the presence of the passive oxide film on steel, unavailability of oxygen and moisture, and high resistivity of concrete, **have become inoperative simultaneously.**

In the context of above-described fundamental principles underlying the process of corrosion of steel in concrete, it may be interesting here to present some recent case studies. Hime and Erlin (34) investigated a structure where steel extending to the ocean surface was found severely corroded at 20-foot depth. Chemical analysis of the rust showed that it consisted of ferrous compounds (ferrous oxide and hydroxide), thus elegantly demonstrating the lack of oxygen at the site of the corrosion. When carried to the surface, the rust warmed the palm of the investigator's hand by readily reacting with the oxygen in the air to form ferric compounds.

In Fig. 7 (top), the authors (34) illustrate why the steel in a **fully submerged reinforced structure** will not corrode, whereas corrosion will occur in the submerged part (at the anode) of a **partially submerged reinforced concrete structure.** Clearly, in the former no cathode could form but in the latter a cathode could form above sea level (where oxygen is available). Fig. 7 (bottom) illustrates the case when a concrete containing epoxy-coated steel was subjected to dynamic loads and seawater. In the cracked structure, corrosion occurred at lugs, but not at the cracked area. Obviously, the cracks established the cathode (oxygen availability). Further according to the authors (34), the anode might have formed due to local inhomogeneities in the steel, perhaps by the presence of inclusions such as manganese sulfide.

The authors (34) point out that most electrochemical corrosion situations present an almost unlimited anode as compared to the cathode, and are thus **cathodically restricted.** Unfortunately, a false understanding of the corrosion mechanism has caused some attempts at preventing corrosion to actually lead to disastrous results. For example, painting a portion of a potentially anodic area can increase the cathode: anode area ratio, promoting severe corrosion. Painting the cathodic area (the area where we wish to restrict the availability of oxygen and water) would be the proper approach to prevent corrosion.

From the above case histories described by Hime and Erlin (34), this author would like to propose that from standpoint of building corrosion-resistant reinforced concrete structures in marine environment, designers should investigate the possibility of isolating the reinforcing steel in the three zones. For instance, instead of a continuous reinforcement extending from the atmospheric zone to the submerged zone, can we provide for overlapping and discontinuous reinforcement in areas where the splashing zone meets the submerged zone? Such discontinuity will make the steel in the submerged zone cathodically restricted permanently.

Brook and Stillwell (35) reported the preliminary results from the **Concrete in the Oceans Program**, started in England in 1976. The results essentially confirm the findings of earlier investigators. For example, field tests on large specimens under exposure conditions typically experienced by North Sea Structures showed that with high-strength concrete (435 kg/m<sup>3</sup> cement content, 0.45 water/cement ratio, 70 MPa 28-day compressive strength) the chloride penetration was limited to a depth of about 30 mm. There was no evidence of corrosion in any of the specimens of reinforced concrete exposed to deep water, however

significant corrosion occurred in the splash zone of only those specimens which had cracks longitudinal to the reinforcement.

In regard to procedures for protection of rebars from corrosion, this author\* stated that the cathodic protection and the use of coated reinforcing bars provide other approaches to the problem, which are costlier than simply using a low-permeability concrete through proper mix proportioning, quality control, and construction practice. In a brief discussion of the various cathodic protection systems, Lehmann (36) states that to control the corrosion of reinforcing steel in concrete although the use of an externally impressed current by bringing the anode-cathode potential difference to zero is relatively new, since the development of conductive coatings in the early 1980's the effectiveness of the cathodic protection technique has been greatly enhanced.

Nevertheless, some of the criticisms on cathodic protection by the RILEM Committee 12-CRC (Corrosion of Reinforcement in Concrete) is still valid (37). The cathodic potential employed may be rather large, consequently in the long term the possibility of hydrogen cracking cannot be ruled out although so far no case has yet been reported. When a current is passed between the metal to be protected as cathode and auxiliary anodes, the passive oxide film on the rebar becomes unstable. Thus it should be noted that besides being a much more sophisticated technique, cathodic protection replaces one very good steel protection mechanism with another. The long-term effects of an externally impressed current on the microstructural changes in concrete that may prove to be harmful for reasons other than steel corrosion (viz, alkali-aggregate activity), are also unknown. Therefore, with new structures, adequate cover of low permeability concrete would be a more satisfactory and cheaper method to protect the steel from corrosion. However, with an existing structure, that has somehow become contaminated with chloride and where significant steel corrosion is already in progress, the cathodic protection offers an effective way to prolong the service life of the structure.

Two types of protection coatings for rebars are in common use: anodic coatings (e.g., zinc-coated steel), and barrier coatings (e.g., epoxy-coated steel). Shimada and Nishi (38) found that after five years of exposure in the splash zone, the zinc coating was damaged by the attack of chloride ions present in seawater. Due to the high cost and concerns regarding the bond strength of epoxy-coated steel and the long-term durability of both zinc-coated and epoxy-coated steel in concrete, the coated rebars have not yet found a general acceptance by the construction industry.

### CONCLUDING REMARKS

From a review of case studies of deteriorated concrete samples in seawater, current design codes for offshore structures, and latest construction in the north Sea, the author has drawn the following conclusions:

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\*Reference 8, p. 158.

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1. With sea structures, the permeability of concrete is the most important factor for long-time durability.
2. To produce a concrete with very low permeability and to maintain the impermeability in service, it is essential to carry out all three of the following: (a) proportion a concrete mixture that will almost become impermeable on curing, (b) adopt good concreting practice, (c) take measures to prevent excessive concrete cracking in service.
3. Very high strength concrete mixtures (60 MPa or higher compressive strength) tend to show exceedingly low permeability ( $< 10^{-14}$  kg/MPa. m.s.). These mixtures invariably contain a high cement content (approx. 400kg/m<sup>3</sup> or more), a low water-cement ratio (max. 0.40), a high-quality coarse aggregate of relatively small size (max. 20 mm), a water-reducing agent (usually a superplasticizer), and a mineral admixture (usually condensed silica fume, however a good fly ash or granulated blast-furnace slag may be substituted in place of silica fume). Additionally, an air-entraining admixture must be used when protection against frost attack is desired.

Such mixtures will no doubt be expensive when compared to ordinary (30-40 MPa) concrete, however in the opinion of this author they will be cost effective, considering the overall initial and life cycle costs. Below 0.40 water-cement ratio, the improvement in durability will far outweigh the improvement in strength with concrete mixtures containing a mineral admixture-superplasticizer combination.

4. Any portland cement corresponding to ASTM Type I specification may be used. In concrete mixtures with a little or no permeability, the chemistry of cement seems to have no effect on the long-time durability of structures exposed to seawater. However, the cement composition should be taken into consideration from standpoint of its effect on the rheological properties of concrete and any adverse cement-admixture interactions. Generally, portland cements with moderate C<sub>3</sub>A (viz., 6-10%) and low-alkali content have worked well with most water-reducing and air-entraining admixtures.
5. To obtain almost an impermeable concrete, the neonatal history of the material must be given as much importance as the mix design. For example, during batching, mixing, transport, placement, and consolidation of the concrete mixture, every effort should be made so that the microstructure of hardened concrete is as homogeneous as possible. Heterogeneities cause stress concentration and later become sites for microcracks. Furthermore, a smooth and impermeable surface finish is important because it will provide the first line of defense against penetration of water and other fluids. Finally, proper temperature and humidity conditions must be maintained for a sufficient period to facilitate cement hydration, which is essential for achieving low permeability.
6. With massive members (600 mm or thicker) made of rich concrete mixtures (high cement content) thermal shrinkage is generally the first cause of concrete cracking unless appropriate measures have been taken to protect concrete from steep thermal gradients. Settlement, overloads (impacts), and premature loading can be other causes of concrete cracking.

7. With reinforced concrete structures exposed to seawater, the cracks and microcracks present in the cover facilitate transport of chlorides, air, and water to the rebar surface. The corrosion of the embedded steel causes expansion and widening of the cracks already present, eventually leading to serious structural damage. Providing adequate cover thickness on reinforcing and prestressing steel is a proven method for reducing the permeability of concrete.

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TABLE 1--CONCRETE-MAKING MATERIALS FOR OFFSHORE STRUCTURES

| Material                           | ACI 357R-84 Recommendation                                                                                                                                                                                                                                        | FIP-1985 Recommendation                                                                                                                                                                                                                                     | North Sea Field Practice                                                                                                                                                                                                                                                                                                                     |
|------------------------------------|-------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|-------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|----------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|
| Cement Composition:                | C <sub>3</sub> A content of portland cement should not be less than 4% to provide protection for the reinforcement. The maximum C <sub>3</sub> A should be limited to 10% to obtain resistance to sulfate attack.                                                 | In the splash zone and atmospheric zone, portland cements with moderate C <sub>3</sub> A content are recommended. Rapid hardening cements should only be used for repair. Low heat of hydration cements are preferred for structures with heavy dimensions. | For Gullfaks C (1986-87), a finer ground portland cement (400 m <sup>2</sup> /kg, Blaine), conforming to ASTM Type II composition (49% C <sub>3</sub> S, 5.5% C <sub>3</sub> A) is used. Concrete for Beryl A (1974) was made with a typical ASTM Type I cement (55% C <sub>3</sub> S, 8% C <sub>3</sub> A, 300 m <sup>2</sup> /kg, Blaine). |
| Admixtures:<br><u>Chlorides</u>    | No chlorides should intentionally be added. Total water soluble chloride ion of a concrete mixture from all the component materials should not exceed 0.1% by weight of cement for normal reinforced concrete, and 0.06% prestressed concrete.                    | CaCl <sub>2</sub> or admixtures containing more than 0.1% chloride by weight of cement should not be used.                                                                                                                                                  | No chloride admixtures are used.                                                                                                                                                                                                                                                                                                             |
| <u>Chemical and air entraining</u> | Where freeze-thaw durability is required, the concrete should contain entrained air as recommended by ACI 201.2R.                                                                                                                                                 | Air entraining agents, workability aids, and retarders are often essential to obtain optimum mix design, but precautions should be taken to evaluate the side effects of each admixture type before use.                                                    | For Gullfaks C, air entrainment was used for concrete in the splash zone. A spacing factor of 0.25 mm was obtained with 3-5% air. The concrete also contains 6 liters/m <sup>3</sup> of a naphthalene sulfonate type superplasticizer.                                                                                                       |
| <u>Pozzolan/c</u>                  | Pozzolans conforming to ASTM C618 (only natural pozzolans and fly ash are covered) may be used provided tests are made to ascertain their relative advantages and disadvantages specially in regard to sulfate resistance, workability, and corrosion of steel.   | High-quality pozzolanic materials, such as special silica fumes may be added to produce improved strength, durability, and workability.                                                                                                                     | Gullfaks C concrete for the splash zone contains approximately 5% silica fume by weight of cement.                                                                                                                                                                                                                                           |
| Aggregates:                        | Natural sand and gravel, or crushed rock conforming to ASTM C33, and lightweight aggregate conforming to ASTM C330. Marine aggregates may be used provided they have been washed to meet the chloride ions limits. No limits on maximum aggregate size are given. | Aggregates likely to undergo physical or chemical changes in concrete to be avoided. Marine aggregates should not be used unless the chloride content is at an acceptable level, and unless the aggregates have a sufficiently low seashell content.        | High-quality natural sand and gravel are used. Concrete in older structures contained up to 32 mm coarse aggregate, but in recently built (1981-87) offshore structures contains 20 mm maximum aggregate size.                                                                                                                               |

TABLE 2--CONCRETE MIX SPECIFICATIONS FOR OFFSHORE STRUCTURES

| Specification                        | ACI 357-84<br>Recommendation                                                                 | FIP-1985<br>Recommendation                                                                                                                                       | North Sea Field<br>Practice                                                                                                                                                                              |
|--------------------------------------|----------------------------------------------------------------------------------------------|------------------------------------------------------------------------------------------------------------------------------------------------------------------|----------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|
| Compressive<br>Strength*<br>minimum: | 35 MPa for all zones;<br>42 MPa where severe<br>surface degradation is<br>likely.            | 32 MPa for all zones;<br>36 MPa where abrasion<br>resistance is required.                                                                                        | Specified strengths for<br>Beryl A (1974) and<br>Gullfaks C (1986-87)<br>concretes were 36 MPa<br>and 56 MPa,<br>respectively. Actual<br>strengths were about<br>20% higher than the<br>specified value. |
| Water/Cement<br>Ratio<br>maximum:    | 0.45 for the submerged<br>zone, and 0.40 for the<br>splash zone and the<br>atmospheric zone. | 0.45 maximum, but 0.40<br>is preferred.                                                                                                                          | 0.41 for Beryl A<br>concrete. 0.38 for<br>Gullfaks C concrete.                                                                                                                                           |
| Cement Content<br>minimum:           | 356 kg/m <sup>3</sup> (600 lb/yd <sup>3</sup> )                                              | 320 kg/m <sup>3</sup> and 360<br>kg/m <sup>3</sup> for 40 mm and<br>20 mm max. aggregate<br>size, respectively. 400<br>kg/m <sup>3</sup> for the splash<br>zone. | 430 kg/m <sup>3</sup> cement was<br>used for Beryl A<br>concrete, and 380<br>kg/m <sup>3</sup> for Statfjord C<br>(1981).                                                                                |
| Consistency                          | no requirement                                                                               | no requirement                                                                                                                                                   | Minimum 120 mm slump<br>was specified for Beryl<br>A concrete. A much<br>higher slump (240 mm)<br>is specified for<br>Gullfaks C due to a<br>25% increase in the<br>steel reinforcement.                 |
| Permeability<br>minimum:             | no requirement                                                                               | no requirement                                                                                                                                                   | 10 <sup>-13</sup> to 10 <sup>-14</sup> kg/Pa.<br>m.s.                                                                                                                                                    |

\*This is 28-d, average compressive strength from 150 mm dia. by 300 mm long standard-cured test cylinders. It should be noted that in the range of 30 to 60 MPa concretes, the cylinder strength, typically, is about 80% of the 150 mm cube strength.

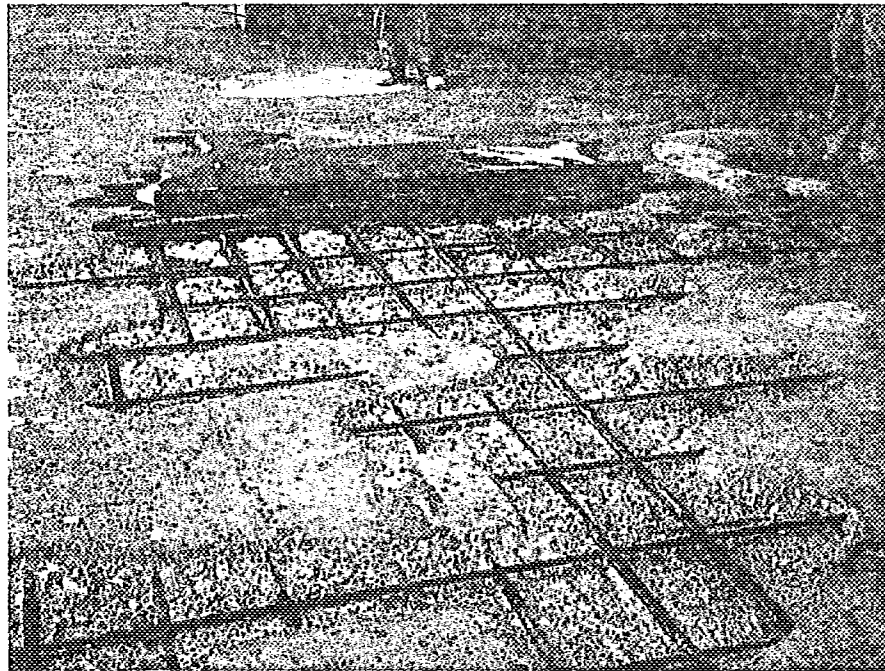


Fig. 1--Photographs showing steel corrosion in structural concrete elements (Hood Canal)

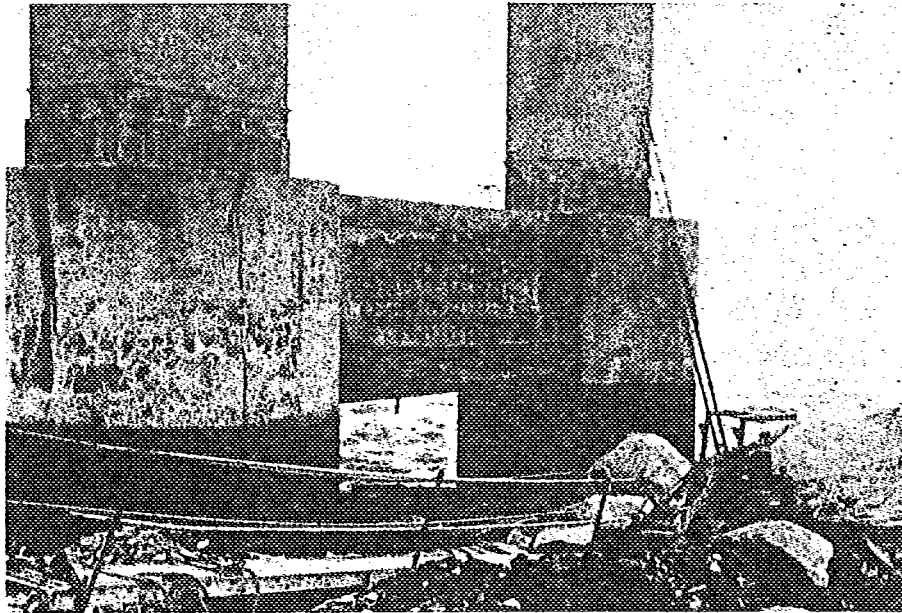


Fig. 2--Photograph showing steel corrosion in spandrel beams (San Mateo-Hayward Bridge)

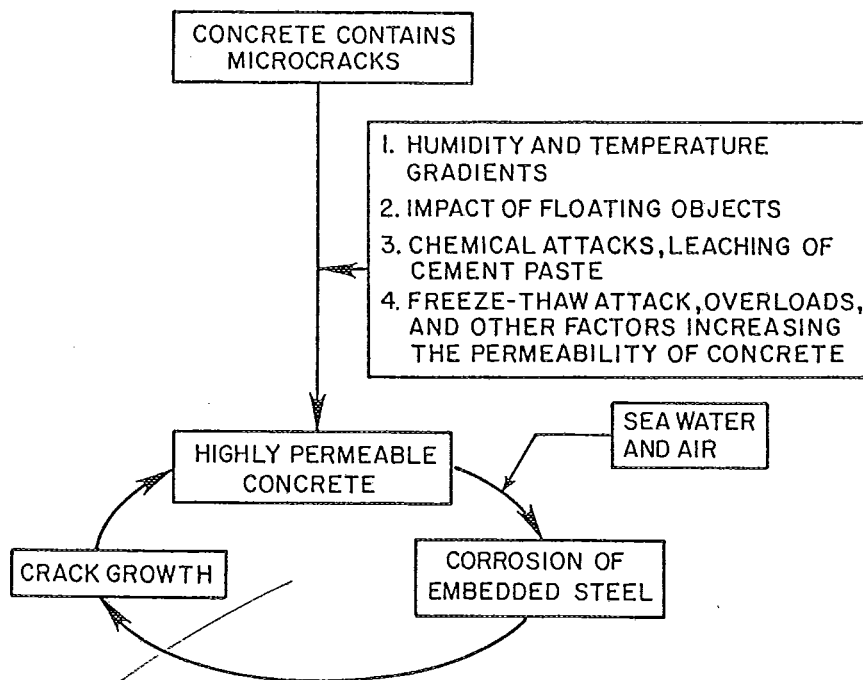


Fig. 3--Diagrammatic presentation of the cracking-corrosion cycle in reinforced concrete exposed to seawater (Reference 2)

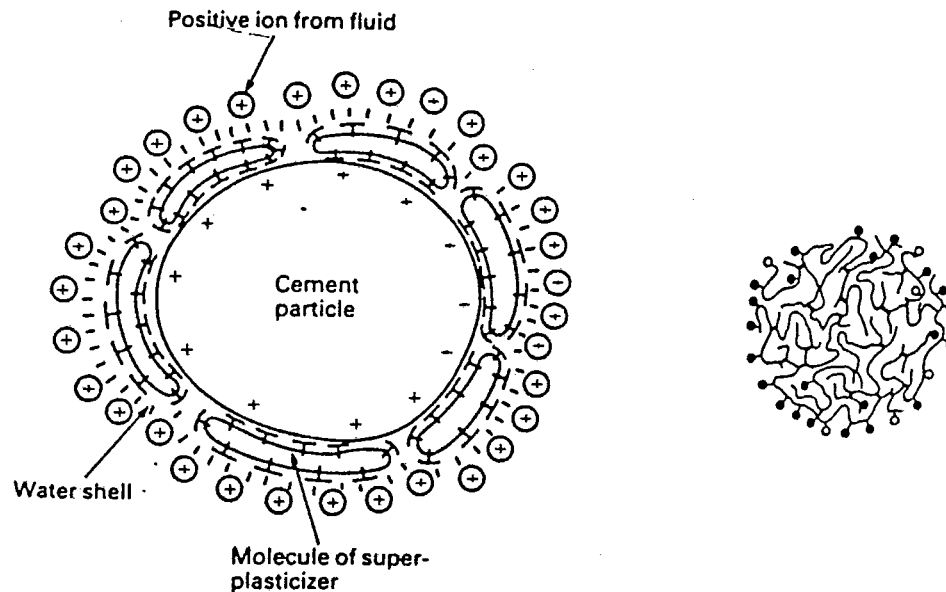


Fig. 4--Effectiveness of linear polymers in coating cement particles (left), compared to cross-linked polymers present in lignosulfonates (right) (from an unpublished report by P.C. Aitcin, University of Sherbrooke, Canada)

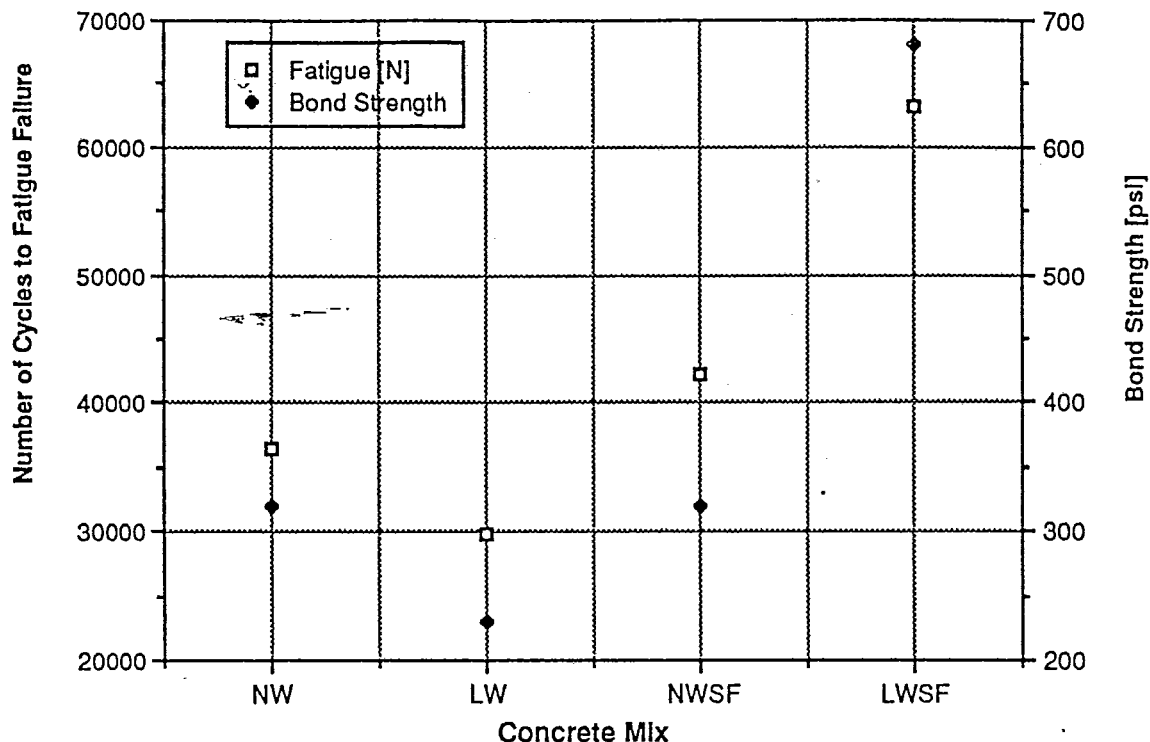


Fig. 5--Fatigue behavior and bond strength of a lightweight silica fume (LWSF) concrete compared to a normalweight silica fume concrete (NWSF) and reference concretes with no silica fume (from Ph.D. thesis of A. Mor, 1986, University of California at Berkeley)

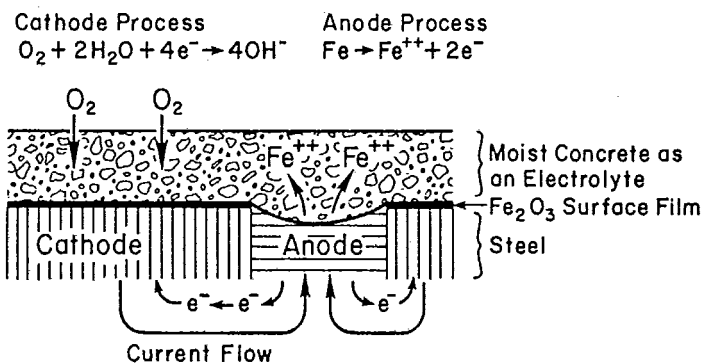


Fig. 6--Diagrammatic presentation of electrochemical corrosion of steel in concrete (Reference 9)

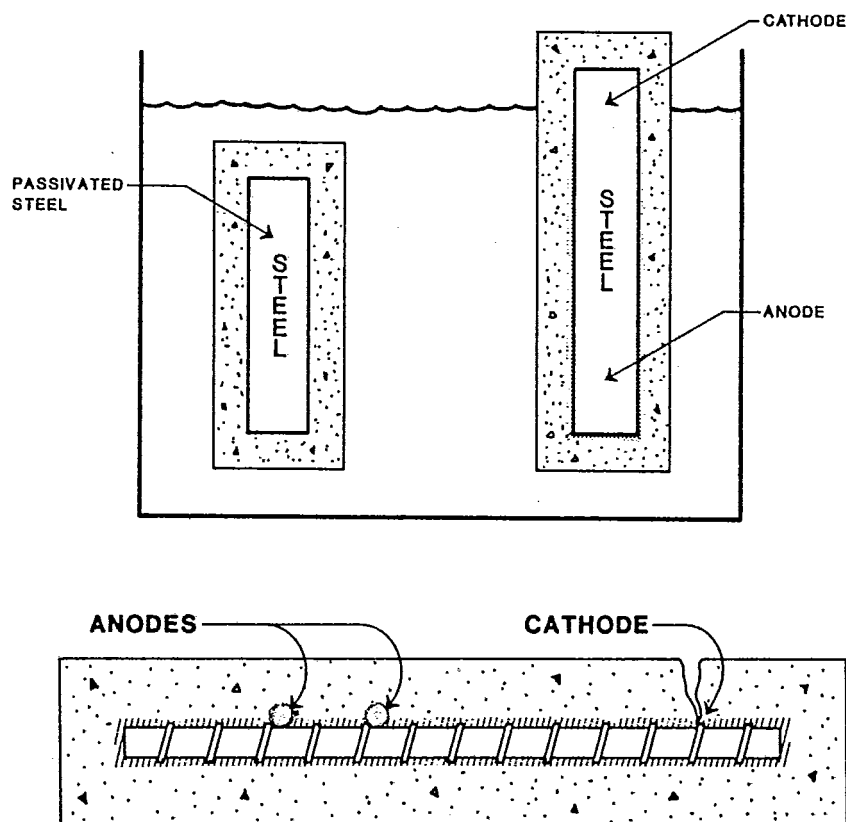


Fig. 7--(top) Examples of steel in concrete, completely or partially immersed in seawater (Reference 34) (bottom) Circled areas represent the site of corrosion of epoxy-coated steel in cracked concrete (Reference 34)

SP 109-2

# Current Status of CANMET's Studies on the Durability of Concrete Containing Supplementary Cementing Materials in Marine Environment

by V.M. Malhotra, G.G. Carette, and T.W. Bremner

**Synopsis:** This paper deals with the evaluation in marine environment of normal and lightweight concretes incorporating supplementary cementing materials. A series of 175 concrete prisms, 305 x 305 x 915-mm in size, were cast over a nine-year period starting in 1978, for long-term exposure at Treat Island, Maine. The prisms were positioned at mid-tide level on a rack at the entrance to the Bay of Fundy, and this represents what is perhaps the most severe marine exposure conditions for concrete. The test specimens are exposed to repeated cycles of wetting and drying, and to an average of about 100 cycles of freezing and thawing per year.

The test specimens are monitored at yearly intervals: the specimens are photographed and rated on a visual basis. Ultrasonic pulse velocity is also determined. After up to 9 years exposure, both normal-weight and lightweight air-entrained concretes show no degradation of the mass of the concrete; however, some of the specimens show significant surface deterioration. The amount of deterioration generally increases with an increasing water-to-cementitious materials ratio, and increasing replacement of cement with slag and fly ash. It appears that surface deterioration can be avoided if the cement content is kept to at least a certain minimum level. The tests confirm that over long exposure duration, non air-entrained concrete is not durable in this environment.

**Keywords:** blast furnace slag; compressive strength; concrete durability; fly ash; freeze-thaw durability; marine atmospheres; portland cements; sea water; silica fume; wetting and drying tests

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## INTRODUCTION

This paper describes the research undertaken by the Canada Centre for Mineral and Energy Technology (CANMET), Energy, Mines and Resources Canada, Ottawa for the determination of durability in marine environment of portland cement concrete incorporating blast-furnace slag, fly ash, silica fume and superplasticizers. The marine environment selected was Treat Island, Maine, an outdoor exposure station operated by the U.S. Army Corps of Engineers. Progress reports covering some early phases of the research have already been published (1,2). This study is a part of a larger project to define the extent to which less energy intensive materials such as fly ash, slag and silica fume can be used to replace portland cement and still maintain an appropriate level of strength and durability (3).

## SCOPE

The test program has been divided into several phases to determine the performance of various types of concrete as follows:

- Phase I: Air-entrained concrete incorporating pelletized blast-furnace slag, and air-entrained and non air-entrained concretes incorporating superplasticizers.
- Phase II: Air-entrained concrete incorporating fly ash and pelletized blast-furnace slag.

- Phase III: Air-entrained semi-lightweight concrete incorporating pelletized blast-furnace slag.
- Phase IV: Air-entrained concrete incorporating fly ash.
- Phase V: Air-entrained concrete incorporating granulated blast-furnace slag, and air-entrained and non air-entrained concretes incorporating silica fume.
- Phase VI: Air-entrained semi-lightweight concrete incorporating fly ash and silica fume (with and without steel fibres).
- Phase VII: Air-entrained semi-lightweight concrete incorporating silica fume (batched at the ready-mix plant).
- Phase VIII: Air-entrained superplasticized concrete incorporating high volume of fly ash.
- Phase IX: Air-entrained concrete incorporating fly ash or pelletized blast-furnace slag or silica fume (with steel reinforcement).

The test prisms from the first phase were installed at Treat Island, Maine in 1978, and the concrete prisms from the subsequent phases were installed at almost yearly intervals, with those from the last phase being installed at the site in 1987. The water-to-cementitious materials ratio of the concrete, in general, ranged from 0.30 to 0.60, and the percentage replacement of cement by supplementary cementing materials ranged from 0 to 80 per cent by weight. A total of 175 prisms, 305 x 305 x 915-mm in size, are being monitored at yearly intervals: the monitoring includes visual rating, non-destructive testing and a photographic record.

#### EXPOSURE CONDITIONS AT TREAT ISLAND, MAINE

In 1936, the U.S. Army Corps of Engineers' Concrete Laboratory for the Passamaquoddy Tidal Power Project established an exposure site at Treat Island in Cobscook Bay near Eastport, Maine. This island is within a few kilometres of the Canadian border and is at the entrance to the Bay of Fundy. At the site, test prisms are positioned on a rack at mid-tide level so that they are exposed alternatively to a marine atmosphere and to immersion in sea water, twice daily. The alternating condition of immersion and exposure provide over 100 cycles of freezing and thawing per year. By using this same facility, deterioration patterns occurring in this study can be related to the behaviour of concretes in investigations performed at Treat Island over the past 50 years.

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## CONCRETE MIXTURES

The concrete mixtures were prepared in the University of New Brunswick Laboratory at Fredericton between 1978 and 1987 using a counter-current pan mixer. The only exceptions were the two mixtures of Phase VII which were prepared at a ready-mixed concrete plant in Saint John, N.B. The details concerning the materials used, mixing procedure and properties of freshly mixed concrete are outlined below.

Materials

Cement--ASTM Types I and V cements were obtained from plants in Havelock, New Brunswick and Brookfield, Nova Scotia. The Type II cement was obtained from a plant in St. Constant, Quebec. The various types of cement used in different phases of the project are shown below.

| <u>Phase</u> | <u>Year</u> | <u>Cement Type (ASTM)</u> |
|--------------|-------------|---------------------------|
| I            | 1978        | I<br>V                    |
| II           | 1979        | I<br>II<br>V              |
| III          | 1980        | I<br>V                    |
| IV           | 1981        | I<br>V                    |
| V            | 1982        | I                         |
| VI           | 1985        | II                        |
| VII          | 1986        | I                         |
| VIII         | 1987        | I                         |
| IX           | 1987        | I                         |

Fly Ash--For investigations of phase II, the low-calcium fly ash (ASTM Class F) was obtained from a plant in Detroit; for investigations of phases IV, VI, VIII, and IX, also a low-calcium ash was obtained from a plant in Lingan, Nova Scotia.

Blast-Furnace Slag—Pelletized blast-furnace slag was obtained from a plant in southern Ontario for the investigations performed in phases I, II, III and IX. The granulated blast furnace slag from a plant at Sparrows Point, New York was used for phase V investigations.

Silica Fume—The silica fume used in phases V, VI, VII and IX investigations was obtained from a silicon and ferrosilicon plant in Becancour, Quebec.

The physical properties and chemical analysis of each of the above materials are given in Tables 1 to 9.

Air-Entraining Admixture—A sulphonated hydrocarbon type air-entraining admixture was used throughout the investigations.

High-Range Water Reducing Admixture (Superplasticizer)—The high-range water reducing admixture (HRWRA) used in Phases VI, VII, VIII and IX and in mixtures SP2 and SP3 of Phase I was a naphthalene-based product of Japanese origin; for specimens SP5 and SP6 of phase I, the HRWRA was a melamine-based product of German origin.

#### Aggregates

| <u>Phase</u> | <u>Year</u> | <u>Type of<br/>Fine Aggregate</u> | <u>Type of<br/>Coarse Aggregate</u>         |
|--------------|-------------|-----------------------------------|---------------------------------------------|
| I            | 1978        | Natural sand                      | Gravel, 37.5-mm max size                    |
| II           | 1979        | Natural sand                      | Gravel, 37.5-mm max size                    |
| III          | 1980        | Natural sand                      | Expanded shale, 25-mm max size              |
| IV           | 1981        | Natural sand                      | Crushed dolomitic limestone, 19-mm max size |
| V            | 1982        | Natural sand                      | Crushed dolomitic limestone, 19-mm max size |
| VI           | 1985        | Natural sand                      | Expanded shale, 19-mm max size              |
| VII          | 1986        | Natural sand                      | Expanded shale, 14-mm max size              |
| VIII         | 1987        | Natural sand                      | Gravel, 19-mm max size                      |
| IX           | 1987        | Natural sand                      | Gravel, 19-mm max size                      |

The gradings of the fine aggregates and the various types of coarse aggregate were within the limits of ASTM Standard C 33.



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Mixture Proportions

The mixture proportions for each phase are summarized in Tables 10 to 18. In general, when supplementary cementing materials were incorporated into the mixtures, these were used as a direct replacement for cement by mass. All mixtures were proportioned to give a slump of  $75 \pm 25$  mm and an entrained air content of  $6 \pm 1$  per cent, except in few instances when non air-entrained concrete was proportioned or when HRWRA were used. When lightweight aggregates were used, these were soaked in water for 24 hours prior to use in concrete. All weights of aggregates are given on a saturated-surface dry basis with the exception of the lightweight aggregates which are on an oven-dry basis.

Properties of Fresh Concrete

The properties of freshly mixed concrete, i.e., temperature, slump, unit weight and air content are given in Tables 19 to 27.

## CASTING AND CURING OF TEST SPECIMENS

The large test prisms (305 x 305 x 915-mm) were cast in two layers of equal depth with each layer vibrated with an internal vibrator. Upon completion of vibration of the filled mould, the excess concrete was struck off with a wooden straight edge. As soon as the bleed water had evaporated, the concrete was floated with a wooden trowel and the concrete covered with a sheet of plastic. The concrete prisms were demoulded after 24 h curing in laboratory air, and then covered with wet burlap and plastic sheeting. After a maximum of three days, the prisms were moved into the moist-curing room where they were stored on their ends. In phase VII where specimens were cast in the field, the prisms were moved indoors after demoulding, and stored under continuously moist conditions.

All prisms cast in 1978 were moist cured for 28 days whereas all subsequent prisms were moist cured at least 90 days prior to being shipped to Eastport for subsequent transport by boat to Treat Island. A view of the concrete specimens installed at Treat Island is shown in Fig. 1.

In addition to the large prisms, a number of 152 x 305-mm cylinders were cast from each mixture for reference (control) strength testing. The specimens were cast according to ASTM procedures and were kept in a moist-curing room until required for testing.

## TESTING OF HARDENED CONCRETE SPECIMENS

Large Prisms Installed at Treat Island

Visual Examination and Rating System--All test prisms are visually examined on an annual basis and evaluated using a rating system shown in Figure 2. The detailed ratings for all specimens of phases I to VII are given in Tables 28 to 34. The ratings for Phases I and II are also illustrated graphically in Fig. 3 and 4.

Pulse Velocity Measurements--Ultrasonic pulse velocity measurements are also being taken on the prisms on an annual basis using a portable digital unit. The velocities are measured at a distance of 50 mm in from the corners and at the centre of the prisms end. Results of pulse velocity measurements obtained over periods up to 7 years are given in Reference 1.

Laboratory Specimens

The 152 x 305-mm cylinders were tested in compression at various ages in the laboratory. The Young's modulus of elasticity of concrete was determined on the cylinders at the age of 28 days. As far as possible, all tests were performed in accordance with ASTM test procedures. The analysis of these test results will be the subject of a separate report and only some selected strength results are presented in this report (Tables 19 to 27).

## DISCUSSION OF PERFORMANCE OF LARGE TEST PRISMS AT TREAT ISLAND

In terms of the durability of concrete, the periods of exposure at Treat Island to this date are insufficient to fully evaluate the long-term relative performance of concrete. However, given the very severe exposure conditions at Treat Island, some trends are clearly apparent in the case of specimens that have been exposed for more than five years, and these are discussed below.

Phase I, 1978

Phase I was primarily concerned with the long-term performance of concrete incorporating pelletized blast-furnace slag (BFS). Percentages by mass of slag examined were 25, 45 and 65 per cent at water-to-(cement + slag) ratios  $[W/(C+BFS)]$  of 0.40, 0.50 and 0.60.

Air-entrained concrete prisms at  $W/(C+BFS)$  of 0.60 and incorporating 65 per cent pelletized slag are showing severe surface deterioration after nine years of exposure and have visual ratings of 4. This implies that prisms with high percentages of the slag used at  $W/(C+BFS)$  of 0.60, even though air-entrained, may not have inherent strength for long-term

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durability under the exposure conditions encountered at Treat Island. In general, there is increasing surface deterioration in all test prisms with increasing water-to-cementitious materials ratio and increasing percentage replacement of cement with slag. Test prisms with a  $W/(C+BFS)$  of 0.40 and incorporating 65 per cent slag show almost no surface deterioration and have a visual rating of 1. Control test prisms made with either Type I or Type V portland cement only, show no deterioration at  $W/(C+BFS)$  of 0.40 and 0.50, however some slight scaling has recently been observed to start at  $W/(C+BFS)$  of 0.60.

Phase I also included a series of test prisms to examine the effect of HRWRA on the long-term performance of both non air-entrained and air-entrained concretes.

As expected, the non air-entrained concrete prisms cast from this series show major deterioration regardless of composition. In fact, prism No. SP6 incorporating HRWRA has completely disintegrated. The air-entrained prisms from the same series (Prisms No. SP1, SP2 and SP5), whether containing HRWRA or not, are all in satisfactory condition and have ratings of no more than 2. This, once again, brings out the importance of air-entrainment for concrete which is to be exposed to repeated cycles of freezing and thawing in saturated environments.

#### Phase II - 1979

Phase II was designed to investigate the performance of concrete incorporating pelletized blast-furnace slag and class F fly ash. The latter was used either as a 25 per cent by mass replacement for cement or in combination with the slag to replace up to 80 per cent of the cement.

Regardless of the water-to-(cement + slag + fly ash) ratio, all test specimens containing 60 per cent pelletized slag and 20 per cent fly ash show extensive deterioration after eight years exposure, and have visual ratings ranging from 4 to 6. This is probably due to the very high percentages of the supplementary cementing materials and low amounts of portland cement used. Somewhat similar deterioration, though to a lesser degree, has been observed for concrete incorporating 40 per cent pelletized slag and 20 per cent fly ash. It appears that for the combination of the materials used in this phase and for satisfactory performance under the exposure conditions at Treat Island, the air-entrained concrete must contain a minimum amount of cement and this appears to be of the order of 200 to 250  $\text{kg/m}^3$ . No significant deterioration has been noticed to date on concrete prisms incorporating 25 per cent fly ash; the visual rating ranges from 0 to 2.

Control test prisms made with ASTM Type I, II and V cements are performing satisfactorily regardless of the water-to-cement ratio, and there is no significant difference in their visual rating.

#### Phase III - 1980

In this phase, the coarse aggregate used was expanded shale and the resulting semi-lightweight concrete had a total cementitious materials content ranging from 240 to 480 kg/m<sup>3</sup> and incorporated up to 65 per cent pelletized blast-furnace slag. The test prisms from series H and I with respective cementitious materials contents of 360 and 480 kg/m<sup>3</sup> do not exhibit any significant change after 7 years of exposure, whereas the test prisms from Series G incorporating only 240 kg/m<sup>3</sup> of cementitious materials show some surface deterioration. Once again, the deterioration is seen to increase with an increasing slag content, and test prisms incorporating 65% slag have a visual rating of 4. The types of cement used do not appear to affect the performance of the concrete. In general, the pattern of deterioration for the semi-lightweight concrete seems to be different from that of normal-weight concrete. With normal-weight concrete, there appears to be a potential for the mortar over the aggregate to come off in a sporadic fashion indicating a plane of weakness at the aggregate-cement paste interface. With lightweight concrete this is not noted, and deterioration occurs by a uniform loss of the surface layer. After six years the lowest strength lightweight concrete prisms show losses of 2 mm or more of mortar which exposes the aggregates in a uniform manner.

#### Phase IV - 1981

This phase had been designed to monitor the long-term performance of concrete incorporating normal-weight aggregates and fly ash from a bituminous source in Nova Scotia. All concrete prisms are in satisfactory condition after six years of exposure, the only exception being the prisms with a water-to-(cement + fly ash) ratio of 0.60 and incorporating fly ash at a replacement level of 25 per cent. The latter specimens have started showing some signs of surface scaling with visual ratings of 3. Once again, there is no apparent difference in the performance of the control concrete made with Type I or Type V cement.

#### Phase V - 1982

This phase involved the monitoring of the long-term performance of air-entrained and non air-entrained concrete incorporating silica fume, and air-entrained concrete incorporating large volumes of granulated blast-furnace slag. After a relatively short exposure of five years, some significant trends are apparent.

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Non air-entrained concrete test prisms made with a water-to-(cement + silica fume) ratio of 0.60 have started showing signs of distress, and a visual rating of as high as 4 has been noted for concrete incorporating 20 per cent of silica fume as replacement for cement. This rating is inferior to that for the corresponding test prisms without silica fume. Somewhat similar results have been observed in laboratory studies at CANMET (4). Relative lack of deterioration observed on other non air-entrained concrete prisms is probably due to the short exposure duration. It is believed that longer exposure time will seriously affect the performance of these prisms. For air-entrained concrete, there appears to be an increase in the visual ratings with increasing amounts of silica fume. However, it is too early to draw any firm conclusions.

The concrete prisms incorporating granulated slag at 80 per cent cement replacement level, and with a  $W/(C+BFS)$  of 0.40 and 0.50, are in satisfactory condition with visual ratings of less than 1. The corresponding prisms with a  $W/(C+BFS)$  of 0.60 have started showing signs of surface scaling with a rating of 2 to 3.

#### Phases VI to IX

Specimens cast under these phases have been exposed for a maximum of 2 years only and therefore no significant trend has been observed yet.

#### General Comments

Pulse Velocity Measurements—The pulse velocity measurements, in general, show either no change or a slight increase with time for all air-entrained concretes. This implies satisfactory condition of concrete with no internal distress or cracking. However, in some instances, there was difficulty in obtaining satisfactory pulse velocity measurements. This was due to poor contact between the concrete and the transducers as a result of deterioration of the surface of the test prisms.

Number of Freezing and Thawing Cycles—The number of freezing and thawing cycles at the exposure site fluctuates from year to year, and the average value for the past nine years is about 80 cycles. However, according to Thornton (5), in 26 winters from 1953 to 1979, the number of annual cycles had ranged from 71 to 185 with the average being 133. Therefore, it is emphasized that due consideration should be given to this aspect when comparing data from different phases.

## CONCLUSIONS

The following conclusions are based upon the exposure of the test prisms at Treat Island for a relatively limited period of time, i.e. up to a maximum of nine years. Some of these will have to be reconsidered after the test prisms have been at the exposure site for 20 to 25 years. Furthermore, these conclusions are applicable only to the brands and types of the materials used in this investigation, and may or may not apply to other types of materials or similar materials from other sources.

Phase I

The surface scaling of test prisms increases with an increase in the water-to-(cement + slag) ratio. At higher ratios, the scaling also increases with increasing percentages of the pelletized slag.

The types of cement used do not affect the performance of concrete investigated.

Non air-entrained concrete is not durable under the exposure conditions experienced at Treat Island.

Phase II

Concrete incorporating high volumes of pelletized slag and fly ash perform relatively poorly. It appears that, for the combination of the materials used in this phase and for satisfactory performance under the exposure conditions at Treat Island, the air-entrained concrete must contain a minimum amount of cement and this appears to be of the order of 200 to 250 kg/m<sup>3</sup>.

There is no significant difference in the performance of concretes made with ASTM Types I, II and V cements.

Phase III

Semi-lightweight concrete made with expanded shale aggregate and having cementitious contents of 360 kg/m<sup>3</sup> or greater perform satisfactorily.

Phase IV

Fly ash concrete at 25 per cent cement replacement level by mass can be satisfactorily used for the exposure conditions investigated, provided that the water-to-cementitious material ratio does not exceed 0.50.

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Phase V

No firm conclusion can be drawn as to the performance of other concrete prisms which have been at the exposure site for a period of less than 5 years. This includes concrete prisms incorporating 80 per cent granulated slag as a replacement for cement and prisms incorporating silica fume.

Phases VI to IX

The very short exposure time for these phases do not allow the drawing of any conclusions at this stage.

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TABLE 1--PHYSICAL PROPERTIES AND CHEMICAL ANALYSES OF CEMENT AND SLAG — PHASE I, 1978

| Description of tests                     | Portland cement       |                       | Pelletized slag        |
|------------------------------------------|-----------------------|-----------------------|------------------------|
|                                          | ASTM Type I           | ASTM Type V           |                        |
| <u>Physical tests - general</u>          |                       |                       |                        |
| Fineness - passing 75 $\mu\text{m}$ %    | 90.7                  | 92.0                  | 92.7                   |
| - passing 45 $\mu\text{m}$ %             | 77.6                  | 85.0                  | 84.2                   |
| - Blaine, $\text{m}^2/\text{kg}$         | 339                   | 323                   | 466                    |
| Setting time (Initial), min.             | 120                   | 330                   | -                      |
| Autoclave expansion %                    | 0.00                  | -0.02                 | -                      |
| <u>Physical tests - mortar strength</u>  |                       |                       |                        |
| Water content, %                         | 48.5                  | 46.0                  | -                      |
| Flow, %                                  | 115                   | 114                   | -                      |
| Compressive strength on 50-mm cubes, MPa |                       |                       |                        |
| 3-day                                    | 22.4                  | 14.4                  | 3.1                    |
| 7-day                                    | 27.2                  | 20.1                  | 8.6                    |
| 28-day                                   | 32.4                  | 31.4                  | 18.0                   |
| <u>Chemical analyses</u>                 |                       |                       |                        |
| $\text{SiO}_2$ , %                       | 20.56                 | 24.04                 | 36.84                  |
| $\text{CaO}$ , %                         | 63.07                 | 64.19                 | 36.92                  |
| $\text{CaO}$ (free), %                   | 0.53                  | 0.23                  | -                      |
| Insoluble, %                             | 0.47                  | 0.23                  | -                      |
| Loss on ignition, %                      | 1.27                  | 0.74                  | 0.23                   |
| $\text{Al}_2\text{O}_3$ , %              | 5.59                  | 3.21                  | 9.53                   |
| $\text{Fe}_2\text{O}_3$ , %              | 1.99                  | 3.85                  | 1.11                   |
| $\text{SO}_3$ , %                        | 4.63                  | 1.94                  | 0.10                   |
| $\text{MgO}$ , %                         | 1.44                  | 1.29                  | 11.07                  |
| $\text{Na}_2\text{O}$ , %                | 0.08                  | 0.13                  | 0.28                   |
| $\text{K}_2\text{O}$ , %                 | 1.27                  | 0.62                  | 0.36                   |
| Alkalies (as $\text{Na}_2\text{O}$ ), %  | 0.92                  | 0.54                  | 0.52                   |
| <u>Compound composition of cement</u>    |                       |                       |                        |
|                                          | <u>C<sub>3</sub>S</u> | <u>C<sub>2</sub>S</u> | <u>C<sub>4</sub>A</u>  |
| Type I                                   | 46.9                  | 23.6                  | 11.4                   |
| Type V                                   | 46.0                  | 34.2                  | 2.0                    |
|                                          |                       |                       | <u>C<sub>4</sub>AF</u> |
|                                          |                       |                       | 6.1                    |
|                                          |                       |                       | 11.7                   |