

Service Life Demonstration Based on Simulated Exposure and Numerical Modelling

by N. R. Buenfeld

Synopsis:

Modelling chloride transport is usually an integral part of demonstrating that a concrete structure in a chloride-rich environments will have the required service life. Ideally this should take account of spacial and temporal variations in exposure environment and concrete properties. This paper describes how this may be approached through a combination of simulated natural exposure and numerical modelling. This approach helps to reduce the errors associated with coupling different transport processes.

The example taken is a reinforced concrete tunnel exposed to chloride-contaminated groundwater. In a laboratory experiment, opposite faces of specimens of tunnel lining concrete were exposed to the maximum hydrostatic head (30m) of groundwater and the minimum relative humidity (35%) expected inside the tunnel and water outflow and chloride accumulation were monitored over a 3 year period. Water vapour diffusion, chloride binding and porosity measurements were made on parallel specimens. The measurements were used as inputs to a numerical model of water and chloride transport to extrapolate from the measured chloride profiles to predict future chloride profiles. The results were very sensitive to ground water chloride concentration and chloride threshold level, but relatively insensitive to hydrostatic head.

Keywords: cement; chloride; concrete; life prediction;
numerical modelling; simulated exposure

2 Buenfeld

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INTRODUCTION

Major civil engineering projects commonly require demonstration that the required service life is highly likely to be achieved. Modelling chloride transport is usually an integral part of this in chloride-rich environments. Fick's 2nd law of diffusion (describing diffusion of an unreactive species into a semi-infinite medium) is commonly used to represent chloride penetration, the variables being the apparent diffusion coefficient (D_a) and surface chloride content (C_s). D_a and C_s values are often determined by curve fitting to chloride profiles measured on existing structures. Even though chloride *does* react with cement paste and diffusion is rarely, if ever, the only transport process in situations where chloride-induced reinforcement corrosion is a concern, this empirical method has its attractions. It is simple to implement and recent developments have included allowing D_a and C_s to be time-dependent and represented by mean, standard deviation and distribution type such that the predictions are probabilistic (1).

However, in situations where transport processes other than diffusion are expected to dominate, where elements are relatively thin (so that the semi-infinite assumption is invalid) or where properties vary with depth, numerical modelling may be more appropriate. A concrete-lined tunnel below the groundwater table with an internal relative humidity of less than 100% is a good example of such a situation and is the case study in this paper.

NUMERICAL MODELLING

Numerical modelling of transport usually involves the concrete member being divided into a series of laminae, each parallel to the exposed surfaces and attributed with relevant physical and chemical properties. Initial boundary conditions are set at the exposed surfaces and the equations governing transport are then solved for each lamina for a small step forward in time; the resulting values are used in the next time increment. In this way, each transport process is modelled individually using continuum mechanics and is then coupled with the other transport processes. The method depends on being able to describe each process by an equation or look-up table involving measurable coefficients.

INGRESS is a finite difference model of water and chloride transport into

concrete developed at Imperial College (2). The main processes modelled and the governing relations and coefficients are listed in Table 1. The corresponding modelling equations and their solutions for different scenarios are presented in Ref. 2. Where possible, numerical solutions have been compared to analytical solutions (2). Matched asymptotics have been used to test the accuracy of the finite difference solutions for wick action (water absorption and water vapour diffusion in series) as this model is too complex to solve using purely analytical techniques (3).

All of the transport processes listed in Table 1 are involved in the common scenario of a concrete lined tunnel with an internal relative humidity of less than 100%, located below the ground water table. In theory, all of the coefficients could be measured separately on concrete specimens and then used as inputs to *INGRESS* to predict overall chloride build up due to the combination of processes. However, experience shows that this is not the best approach. First, not only are there at least 6 coefficients required, but several of these vary with time and so a very large number of measurements would be needed. Second, the main assumption in a numerical model of this kind is that transport by the different processes occurs to the same extent in the same porosity. This is almost certainly not the case, with transport by convection not occurring to the same extent as diffusion in the smaller pores. Where time is available and the exposure environment is appropriate, a more effective approach, which is demonstrated here, is to apply simulated natural exposure to specimens in the laboratory, monitoring water and chloride transport and then use the numerical model to extrapolate forward in time based on a more limited number of additional measurements.

APPLICATION

Tunnel construction and exposure

The tunnel (which cannot be named to retain confidentiality) is formed from 250 mm thick precast concrete segments, with steel reinforcement protected by 40 mm of concrete cover at each face. The concrete has a binder content of 400 kg/m³ of which 30% is fly ash and a free water-cementitious materials ratio of 0.34. The segments are exposed to up to 30 m head of groundwater containing typically 2 g/l chloride and up to 3.5 g/l chloride. The air inside the tunnel is at around 20 degC and 35% relative humidity, whilst the temperature of the ground is around 16 degC. All of the processes listed in Table 1 are expected to contribute to, or influence, chloride penetration.

Measurements

At an age of 28 days, 100 mm diameter 250 mm long cylinders of tunnel

4 Buenfeld

lining concrete were vacuum saturated with distilled water (to provide a more clearly defined starting moisture state) and were then subjected to simulated natural exposure. This involved applying 30 m head of chloride contaminated groundwater (3.5 g/l chloride) to the extrados (external surface) and monitoring the outflow of water at the intrados (internal surface), which was held at 35% RH. The results that follow are for one particular specimen. The outflow of water with time is shown in Fig. 1.

It can be seen that the rate fell during the test. Over the 1000 days of exposure there was a total outflow of only 21.3 g and parallel tests showed that a major contribution to the early water outflow was drying of the concrete close to the intrados with the total inflow being only around 10.3 g. By the end of the test the flow rate was around 0.006g/day. If this flow rate is attributed entirely to the pushing effect of 30 m head of groundwater, with no pulling contribution from wick action (4), this final flow rate equates to a coefficient of permeability of 1.84×10^{-14} m/s, an extremely low value. In fact, wick action will have contributed to flow and so the permeability will be even lower than this.

After simulated exposure, the cylindrical specimen was subjected to a range of tests. The total chloride profile was measured and is shown by the closed symbols in Fig. 2. Chloride penetrated to a depth of only around 10 mm: the hump at 9 mm is probably the result of a higher cement paste content at this location. Discs cut parallel to the intrados face were tested using 3 gravimetric methods to estimate the porosity contributing to transport; methanol exchange porosity ranged from 5.0 to 5.3% and the empty porosity after drying to 35% relative humidity was around 4%. The chloride binding isotherm, shown in Fig. 3, was measured using the equilibrium method (5). The x-axis of Fig. 3 represents the free chloride content of both the groundwater and of the concrete pore solution.

Modelling

INGRESS was used to fit a curve to the 1000 day chloride profile measured in the simulated exposure test (Fig. 2). INGRESS inputs were lining thickness (250 mm), applied head (30 m), groundwater chloride concentration (3.5 g/l), water inflow vs time (Fig. 1 - note this results from pressure-induced water flow and wick action), effective porosity (5%), chloride binding isotherm (Fig. 3) and background chloride content (0.003% chloride by weight of concrete).

Chloride diffusion coefficient was used as the fitting variable such that the total amount of chloride predicted as entering the concrete was equal to the measured amount, less the background content. This resulted in a diffusion coefficient of 3×10^{-12} m²/s. One of the reasons that it was chosen for fitting was that recent research shows that the chloride diffusion coefficient measured using conventional methods is subject to errors, that are currently impossible to accurately quantify, due to membrane potential effects (6). It was therefore

necessary to quantify chloride diffusion under conditions as closely simulating the exposure environment as possible.

The fitted profile is shown by the open symbols in Fig. 2. The surface chloride content in the fitted profile is entirely dependent upon the chloride binding isotherm and is slightly lower than the measured value. This was expected and is a consequence of the higher cement paste content of the surface concrete in relation to the bulk.

The fitted profile for 1000 days of exposure formed the basis for the prediction of future chloride profiles. To be conservative (safe) it was assumed that the water inflow and chloride diffusion coefficient remained constant at their 1000 day values of 0.006 g/day and $3 \times 10^{-12} \text{ m}^2/\text{s}$, rather than reducing with further hydration. The same porosity (5%) and chloride binding isotherm (Fig. 3) were also assumed and predictions were based on the 250 mm thick section being divided into 250 one mm thick elements.

Fig. 4 shows the predicted effect of groundwater chloride concentration on chloride profiles at 120 years, the required service life of the tunnel. It can be seen that chloride penetration is confined to within 100 mm of the extrados surface such that both chloride precipitation at the position of water evaporation near the intrados and release of bound chloride due to carbonation at the intrados can be ignored.

Fig. 5 shows the predicted effect of hydrostatic head on chloride profiles at 120 years, making the assumption that all of the water flow is associated with wick action. Despite this assumption, it can be seen that the effect of head is very slight. At first sight this may be surprising, but these results are consistent with the simulated exposure results which showed this concrete to have extremely low permeability.

Time to corrosion

To predict the time to corrosion of embedded steel requires information concerning the cover depth and the amount of chloride necessary to initiate corrosion, the chloride threshold level (CTL). The tunnel linings under investigation have reinforcing steel in both the intrados and extrados faces with a nominal cover depth of 40 mm. Chloride will reach the extrados steel first, but oxygen is also necessary for corrosion. If the extrados steel is electrically insulated from the intrados steel, it is likely that there would be insufficient oxygen available at the extrados steel to sustain a significant rate of corrosion and so chloride accumulation at the intrados rebar (which is embedded in partially saturated and therefore oxygen accessible concrete) would control corrosion and extremely long service lives would be predicted. In the case under consideration the intrados and extrados rebars are electrically connected and so oxygen at the intrados steel can drive corrosion of the extrados steel and it is chloride accumulation at the extrados rebar that will control corrosion. Hence the relevant cover depth is 40mm.

6 Buenfeld

The most appropriate means of expressing the CTL is in terms of total (not free) chloride content (7.8). A wide range of CTL values have been reported in the literature, ranging from around 0.2 to 2.5% chloride by weight of cement (including replacements). This wide range can largely be explained by variations in the entrapped air content at the steel concrete interface (9). The lower the entrapped air content at the steel, the higher the chloride threshold level and the more corrosion resistant the lining. For design purposes (i.e. providing a margin of safety), 0.4% is the value most commonly adopted. For concrete which has a total cementitious content of 400 kg/m³ and a density of around 2,400 kg/m³, this is equivalent to 0.067% chloride by weight of concrete. On this basis, Fig. 3 can be used to predict the CTL above which the linings are immune to chloride-initiated reinforcement corrosion until, in the extremely long-term, the chloride concentration in the concrete pore water exceeds that in the groundwater due to back-diffusion from the point of wick action evaporation (3). For example, it can be seen in Fig. 3 that at a groundwater chloride concentration of 4 g/l the maximum achievable chloride content (without extremely long-term wick action effects) would be around 0.17 % chloride by weight of concrete (equivalent to 1.0% chloride by weight of cement), while this figure falls to around 0.09% at a groundwater chloride concentration of 2 g/l.

Fig. 4 includes lines representing the cover depth (40 mm) and CTL (0.067%). It can be seen that for a groundwater chloride concentration of 4 g/l (with 30 m head), the CTL of 0.067% chloride by weight of concrete is predicted to have penetrated to a depth of around 38 mm after 120 years of exposure. A mean time to corrosion of 133 years is predicted for a cover of 40 mm. Of course concrete properties, cover depth and environmental characteristics are stochastic variables and ideally would have been represented by statistical distributions to enable the probability of corrosion initiation at different times to be assessed; however, in this instance there was insufficient data for such an analysis.

CONCLUSIONS

- The empirical application of Fick's second law is not appropriate for modelling chloride penetration in situations where transport processes other than diffusion dominate, where elements are thin in relation to the depth of chloride penetration, or where transport properties vary with depth.
- A numerical model using continuum mechanics and measurable physical and chemical properties allows the effects of several transport processes to be combined, allows realistic boundary conditions to be applied, and allows concrete properties to be varied with time and depth.
- Numerical modelling usually requires a large amount of transport coefficient data for the concrete of interest. The number of measurements can be reduced

by applying simulated natural exposure to specimens in the laboratory, monitoring the associated transport, and then using a numerical model to extrapolate forward in time based on a more limited number of additional measurements. This approach should also reduce errors arising from assumptions regarding the coupling of processes.

Acknowledgements

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8 Buenfeld

Table 1. Processes contributing to, or influencing, chloride transport in concrete

Process	Governing relation	Coefficients
Chloride ion diffusion	Fick's 1 st law	Chloride diffusion coefficient (m^2/s) Porosity (volume fraction)
Chloride binding	Binding isotherm	Bound vs free chloride plot
Pressure-induced water flow	Darcy's law	Water permeability (m/s)
Water absorption	Sharp wet front penetrating as $\text{time}^{1/2}$	Sorptivity ($\text{m/s}^{1/2}$)
Water vapour diffusion	Fick's 1 st law	Water vapour diffusion coefficient (m^2/s)

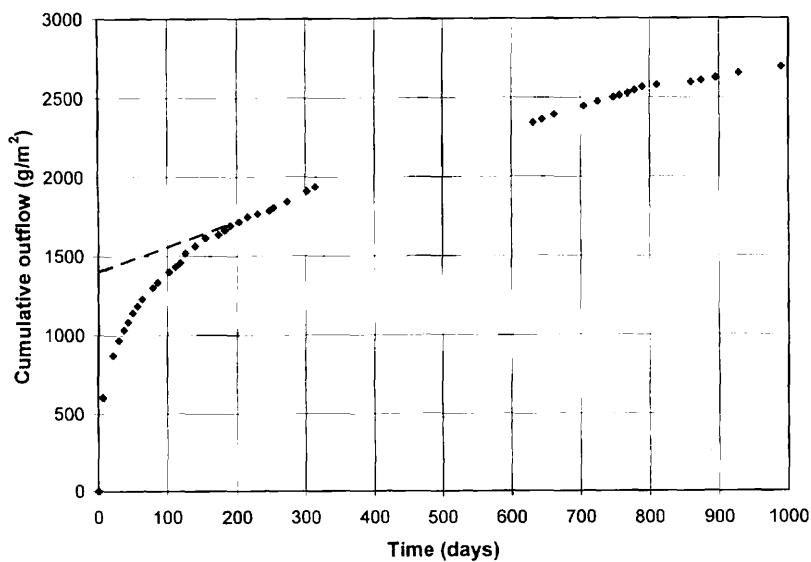


Fig. 1. Cumulative outflow of water vs. time

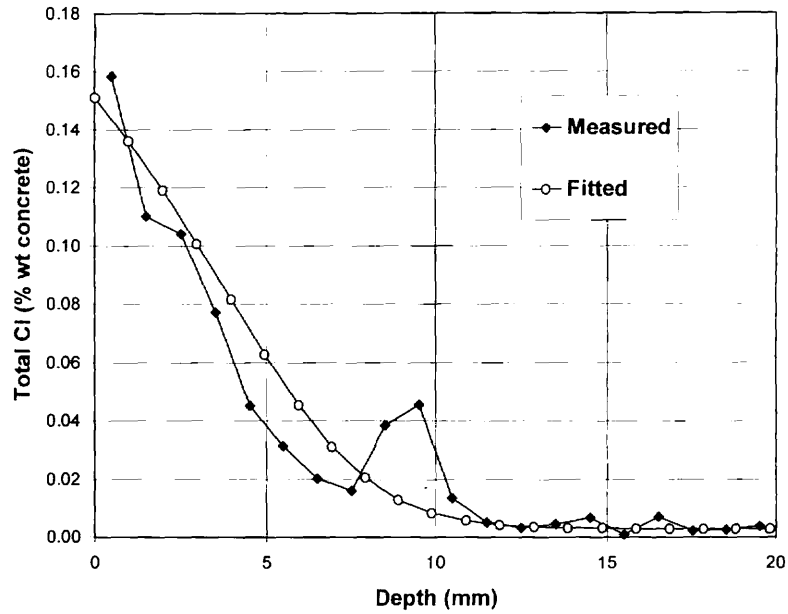


Fig. 2. Measured and fitted chloride profiles (3.5 g/l chloride in groundwater)

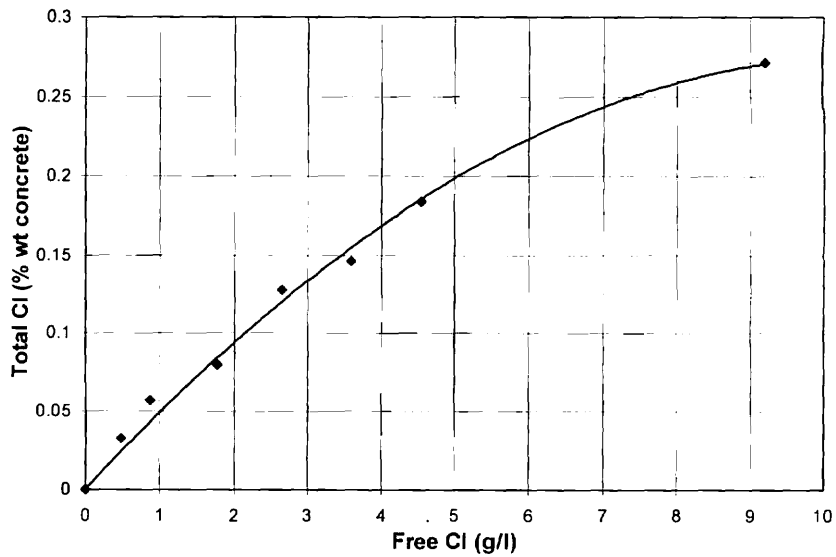


Fig. 3. Measured chloride binding isotherm

10 Buenfeld

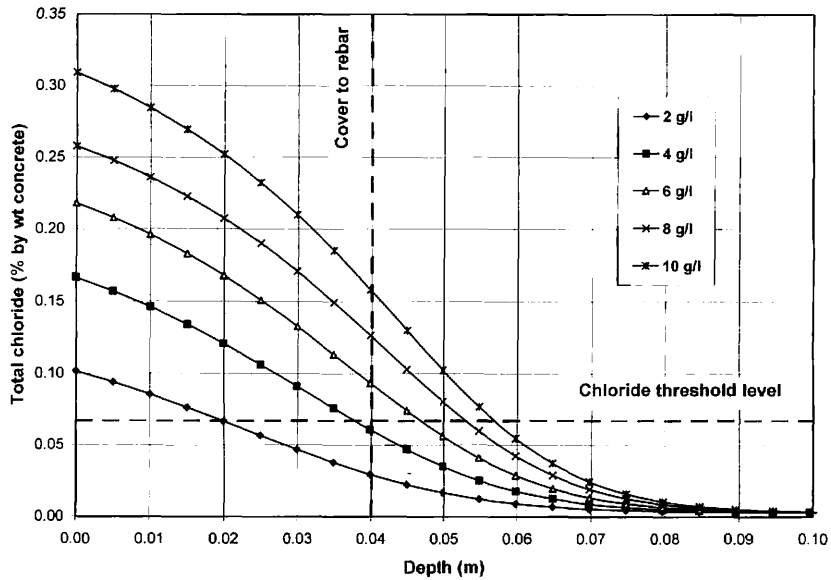


Fig. 4. Predicted effect of chloride concentration on chloride profiles after 120 years of exposure (30m head).

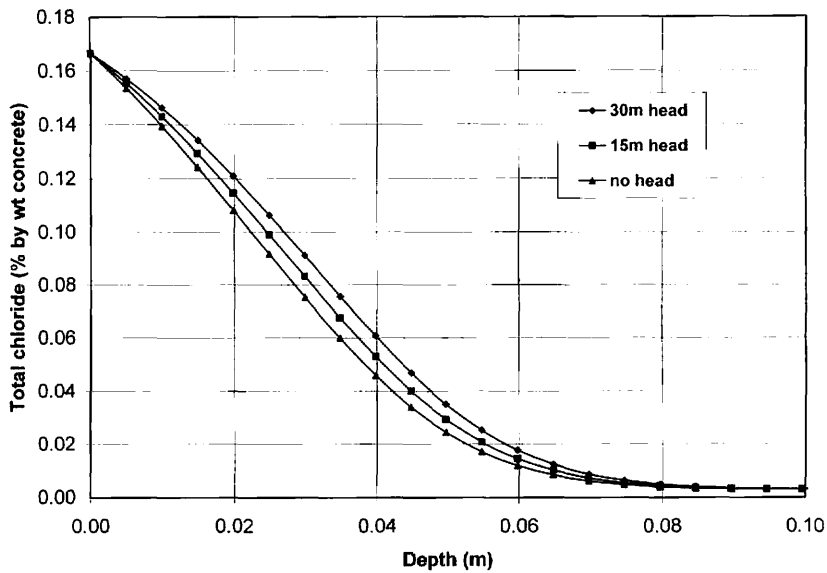


Fig. 5. Predicted effect of hydrostatic head on chloride profiles after 120 years of exposure (4.0 g/l chloride).

Low-Heat Development in Flowing Concretes for Massive Structures

by A. Borsoi, S. Collepardi, R. Troli, and M. Collepardi

Synopsis:

The purpose of this paper is to study the production of crack-free massive concrete structures manufactured by using flowing mixtures characterized by low heat development and the following performances of the mixtures:

- compressive strength in the range of 30–40 MPa at 28–90 days;
- slump level of about 200 mm.

The main ingredients of these mixes were:

- normal portland Cement (not more than 150 kg/m³);
- fly ash as pozzolanic mineral addition (up to 150 kg/m³);
- ground limestone as filler (70–250 kg/m³);
- superplasticizers based on acrylic or sulfonated naphthalene polymers;
- natural aggregates with a maximum size of 20 mm.

Keywords: flowing concrete; heat of hydration;
massive concrete structures; superplasticizers

12 Borsoi et al.

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INTRODUCTION

For the purposes of this paper, mass concrete is a special cement mixture devoted to structures with massive dimensions, such as a gravity dam or, in general, any other concrete member of such dimensions that the thermal behavior may cause cracking unless appropriate measures are taken (1).

Due to the heat of hydration, there is a rise in the temperature of concrete. This rise is not uniform since the heat loss to the ambient atmosphere is higher from the surface of the concrete member than from its interior (Fig. 1). When the temperature difference (δT) between the surface and the interior of the same member is greater than 20°C approx. there is a risk of external or internal cracking (2, 3). The cracking is external when this thermal difference occurs during heating (δT_h) and internal when it occurs during cooling (δT_c), both, however, occurring within 2-3 days from the time (t) of placing (Fig. 1).

PARAMETERS AFFECTING THE THERMAL DIFFERENCE

The thermal difference (δT) between the nucleus of the structure and its surface depends on many parameters not related to the concrete composition such as:

- thermal and windy conditions of the environment;

- presence or absence of the formwork on the surface of the concrete member;
- thermal conductivity and thickness of the formwork when present;
- geometry and size of the concrete members.

Due to the numerous combined changes of the above parameters, it is not easy to predict theoretically how δT will change as a function of the time t in Fig. 1 or how that change will affect the cracking potential. However, this prediction can be done for one case when extreme ideal conditions would occur such as the following ones:

- no heat loss in the nucleus of the concrete member where the temperature would rise according to the heat development of cement hydration and the concrete specific heat;
- complete dissipation of the heat of cement hydration so that there is no temperature rise on the surface.

In these extreme conditions, the adiabatic temperature rise (ΔT_t) of the nucleus at the time t coincides with the maximum thermal difference $(\delta T_t)_{\max}$ between the nucleus and the surface which remains always at the same original temperature, T_o (Fig. 2). The ΔT_t value can be calculated through the equation [1]:

$$Q_t = m \cdot \rho \cdot \Delta T_t = m \cdot \rho \cdot (\delta T_t)_{\max} \quad [1]$$

where Q_t is the total heat developed by cement hydration at the time t in the mass m of concrete, and ρ is the specific heat of the concrete (about $1.1 \text{ kJ} \cdot \text{kg}^{-1} \cdot \text{C}^{-1}$).

If the mass (m) is referred to that of 1 m^3 of concrete, then equation [1] becomes:

$$Q_t = q_t \cdot c = m \cdot \rho \cdot \Delta T_t = m \cdot \rho \cdot (\delta T_t)_{\max} \quad [2]$$

where q_t is the unitary heat developed by 1kg of cement at the time t , c is the cement content in 1 m^3 of concrete, and m becomes the specific mass of the concrete (about 2400 kg/m^3).

Equation [2] can also be written as:

$$(\delta T_t)_{\max} = \Delta T_t = \frac{q_t \cdot c}{2400 \cdot 1.1} = \quad [3]$$

According to the equation [3], the temperature rise (ΔT_t) of the nucleus of the concrete member, and therefore the maximum thermal difference $(\delta T_t)_{\max}$ in the same concrete member at the same time t , can be reduced to a level lower than the critical value of 20°C (2,3) provided that:

$$20 > \frac{q_t \cdot c}{2400 \cdot 1.1} \quad [4]$$

and, then, provided that

$$20 \cdot 2400 \cdot 1.1 = 52,800 > q_t \cdot c \quad [5]$$

This means that, in order to avoid the risk of thermal cracking in a concrete member (Fig. 1), a cement with a low unitary heat of hydration (q_t) and/or a low cement content (c) should be adopted so that:

$$52,800 > q_t \cdot c \quad [6]$$

Since, in practice, the risk of cracking ($\delta T > 20^\circ\text{C}$) occurs at about 3 days from placing concrete (Fig. 1), we can realistically adopt 3 days for the time t in the equation [6]:

$$52,800 > q_3 \cdot c \quad [7]$$

SCOPE OF THE WORK

The purpose of the present paper was to study whether or not mass concrete members, not affected by thermal cracking because there is agreement with equation [7], might be produced even when they are manufactured as flowing mixtures at a slump level of about 200 mm.

Flowing concretes need, in general, a relatively high amount of mixing water (w) and then a relatively high cement content (c) for a given w/c ratio. Moreover, a reactive cement, and then a relatively high heat development at the early ages (q_1 - q_3), is needed when a relatively high compressive strength (30-40 MPa) is required. These requirements for flowing concretes are just the opposite for the mixtures of most massive concrete members which need a cement content as lowest as possible and use a less reactive cement to reduce the heat of hydration at early ages.

EXPERIMENTAL

Materials: Table 1 shows the physico-chemical characteristics of the high strength normal portland cement (*PC*), fly ash (*FA*) used as pozzolanic mineral addition, and the ground limestone (*GL*) filler used in different proportions for the concrete mixtures.

The particle size distribution of the combined aggregate (natural sand and 8-20 mm gravel) is shown in Fig. 3.

Concrete Mixtures: All the concrete mixtures were manufactured with an aggregate/*PC* ratio of 12.7 and a water/*PC* (*w/c*) of 0.93.

The *PC* content was limited to about 150 kg/m³ so that, by taking into account its heat of hydration (Table 1) developed at three days ($q_3 = 350$ kJ/kg), the upper limit of equation [7] is not exceeded:

$$52,800 > 350 \cdot 150 = 52,500$$

The dosage of superplasticizers (4), based on acrylic polymer (*AP*) or sulphonated naphthalene polymer (*SNP*), was adjusted in the range of about 0.3-0.7% or 0.7-1.1%, respectively, by mass of binder (*PC+FA*).

In order to reduce bleeding and segregation of the fresh mixtures, as well as to increase the long-term strength of the hardened concretes, various amounts of fly ash were used (0-50-100-150 kg/m³). Due to the different amount of fly ash, the water/binder ratio – where the binder include both *PC* and *FA* – was in the range of 0.93-0.47 (Table 2).

Moreover, Table 2 indicates that the presence of fly ash (at least 50 kg/m³ as in mixes *B* and *B'*) is needed to avoid a shear slump of the flowing concretes recorded in the absence of fly ash (mixes *A* and *A'*). From this point of view, flowing mixtures with a slump of at least 200 mm, but at the same time cohesive, unsegregable and bleeding-free such as the concretes *D* and *D'* contain about 150 kg/m³ of *PC*, 150 kg/m³ of *FA* and 70 kg/m³ of *GL* filler.

Properties of Hardened Concrete: Two aspects were checked in the hardened state of the concrete mixes shown in Table 2:

- temperature-time trend such as that shown in Fig. 1 in the interior and on the surface of mass concrete members (Fig. 4-5);
- compressive strength from 1 to 90 days at 20°C (Fig. 6-7);
- permeability to water measured through the penetration of water under pressure (up to 7 bar) according to the ISO DIS 7031 test.

Figure 4 shows a concrete block (500x500x500 mm) thermo-insulated by a thick layer of light polystyrene. Two thermocouples, one immersed in the nucleus and the other put on the surface of the member, were used. The highest and the lowest temperature rise, respectively, were measured as a function of the time after placing (Fig. 5).

The data of Fig. 5 are related to the mixture *D'* shown in Table 1. Similar results, not shown here, were obtained with all the other mixtures of Table 1. The data of Fig. 5 indicate that in these quasi-adiabatic conditions for the interior of the concrete block, very close to those illustrated in Fig. 2 for the

temperature rise in the interior, the maximum temperature rise (ΔT_n) in the nucleus was less than 16°C at about 2 days. The temperature rise on the surface was about 15°C, and then a δT_2 of 1°C (16°C-15°C) was recorded. This low value in the thermal difference between nucleus and surface (δT_2) is related to the thermo-insulation of the concrete block of Figure 4. If the dissipation of the heat from the surface were complete, then a thermal difference of less than 16°C, lower than the critical value of 20°C for thermal cracking, would be recorded.

Figures 6 and 7 show the compressive strength trend, from 1 to 90 days, for superplasticized flowing concretes with the *AP*-based and *SNP*-based admixtures, respectively. No significant difference has been recorded for the strength as a function of the superplasticizer type (*AP* or *SNP*).

The concrete compressive strength is strongly dependent on the $W/(PC+FA)$ ratio and at a given time, specially after 7 days, it increases by reducing the $W/(PC+FA)$ ratio. At 90 days, the compressive strength increases by about 50% by passing from the mixtures *A* and *A'* without fly ash, to the mixtures *D* and *D'*, respectively, where the fly ash content was about 150 kg/m³ (Table 1).

Last, but not least, mixes *D* and *D'*, (both with about 150 kg/m³ of normal Portland Cement, 150 kg/m³ of fly ash and 70 kg/m³ of ground limestone as filler) were impermeable according to ISO DIS 7031 test where the penetration of water into the concrete specimens were 6 and 7 mm against the critical value of 20 mm accepted for impermeable concretes.

CONCLUSIONS

The combined use of a high strength normal portland cement (150 kg/m³), fly ash (150 kg/m³) as pozzolanic addition, ground limestone (70 kg/m³) as filler, and natural aggregates with a maximum size of 20 mm allow:

- the manufacture of flowing concretes (slump $\geq$ 200 mm), without segregation, very cohesive even in the presence of the limited amount of portland cement (150 kg/m³);
- limiting the temperature rise in the nucleus of a massive concrete block in quasi-adiabatic conditions to less than 16°C, then with a thermal difference (δT) between the nucleus and the surface much lower than the critical value of 20°C adopted for the absence of thermal cracking in massive concrete structures;
- compressive strengths in the range of about 25-35 MPa at 28 days and 30-40 MPa at 90 days.

All of the above results appear to be, to a certain extent, revolutionary since superplasticized flowing concretes, with a relatively small aggregate (maximum

size of 20 mm), could be placed in massive concrete structures, including a gravity dam, without any risk of thermal cracking and relatively high strength up to 35 and 40 MPa at 28 and 90 MPa, respectively.

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18 Borsoi et al.

Table 1 – Physico-chemical characteristics of Portland Cement (PC), Fly Ash and Limestone

Mineralogical Composition (%)	PC*	Fly Ash**	Ground limestone
C ₃ S	60	-	-
C ₂ S	20	-	-
C ₃ A	12	-	-
C ₄ AF	3	-	-
CaSO ₄ 2H ₂ O	5	-	-
CaCO ₃	-	-	96.8
Blaine Fineness m ² /kg	540	310	470
Heat of hydration (kJ/kg) at:			
1 day	280	-	-
3 days	350	-	-
7 days	400	-	-
28 days	440	-	-

*CEM I 52.5R according EN 197/1 ; ** SiO₂ = 60%

Table 2 - Composition of concrete mixes and properties in the fresh state

MIX	PC	FA	GL	Superplasticizer		w/c	$\frac{w}{c + fa}$	Slump (mm)	Specific Mass (kg/m ³)	Cohesiveness by Visual Assessment
				Type	(%)*					
A	149	0	252	SNP	0.72	0.93	0.93	200	2437	Shear Slump
B	148	50	169	SNP	0.85	0.93	0.70	200	2456	Good
C	146	100	131	SNP	0.99	0.93	0.56	220	2446	Very Good
D	145	151	69	SNP	1.13	0.93	0.47	210	2437	Excellent
A'	149	0	252	AP	0.28	0.93	0.93	200	2437	Shear Slump
B'	148	49	189	AP	0.42	0.93	0.70	200	2408	Good
C'	146	98	128	AP	0.55	0.93	0.56	220	2379	Very Good
D'	145	145	67	AP	0.68	0.93	0.47	200	2340	Excellent

* by weight of Portland Cement and fly ash

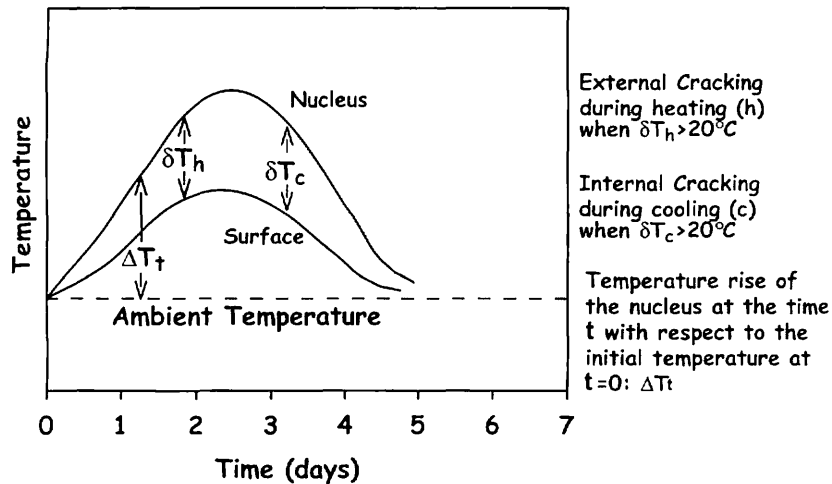


Fig. 1 – Schematic example of temperature change in the nucleus and on the surface of a concrete member with respect to the ambient temperature.

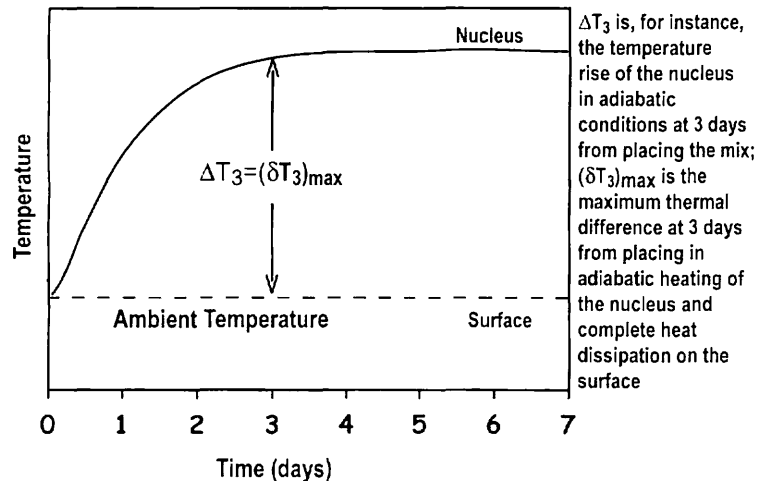


Fig. 2 – Schematic example of temperature change in the nucleus in adiabatic conditions and on the surface of a concrete member with complete heat dissipation.

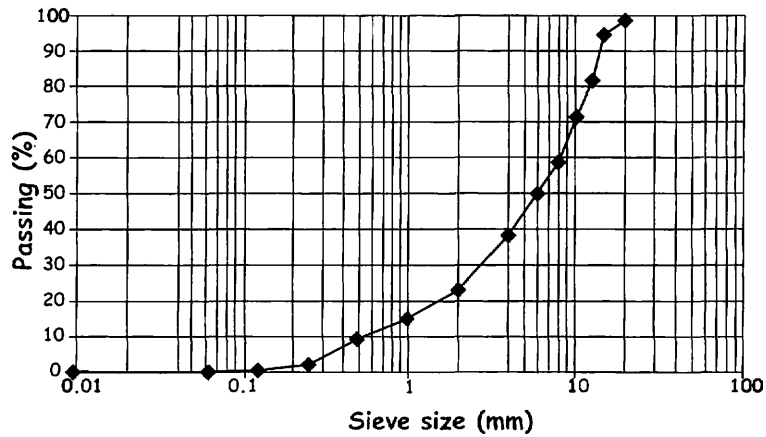


Fig. 3 – Particle size distribution of combined aggregate.

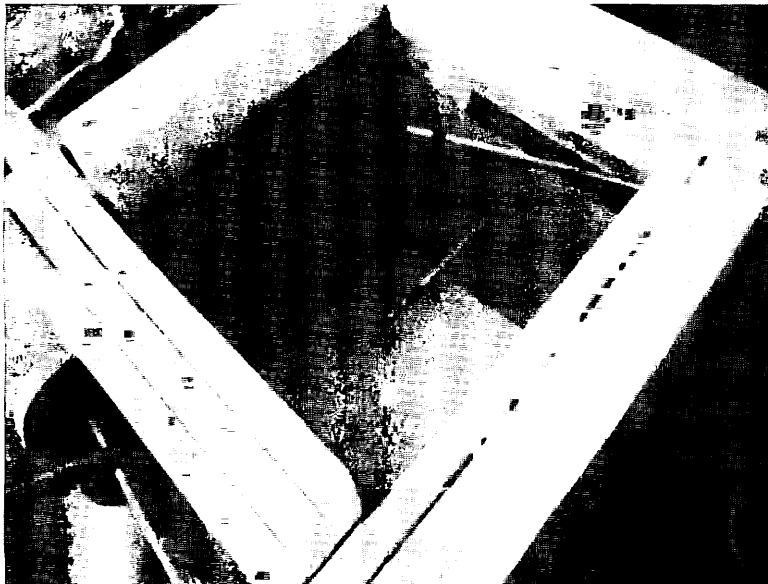


Fig. 4 – Thermo-insulated formwork for the concrete block (500x500x500 mm) with thermocouples to record the temperature rises shown in Fig. 5.

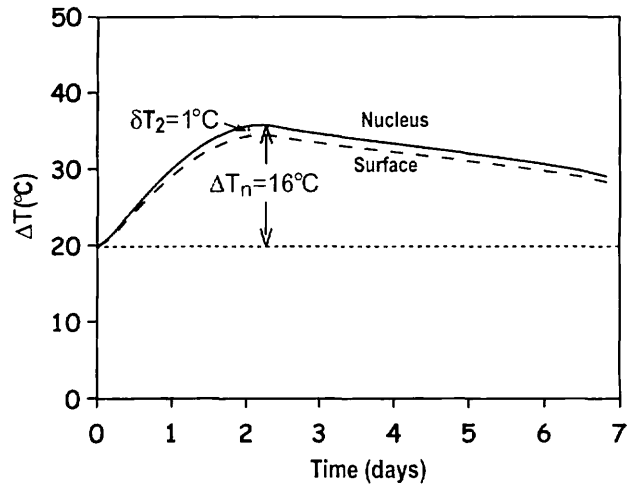


Fig. 5 – Temperature rise, ΔT , in the nucleus and on the surface with respect to the initial temperature of 20°C in quasi-adiabatic conditions.

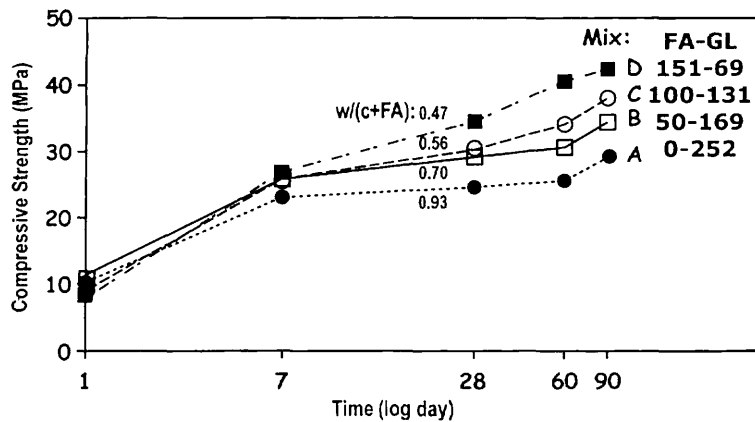


Fig. 6 – Compressive strength as a function of the time for concrete mixtures A, B, C and D of Table 1 (superplasticizer: *SNP*). FA and GL indicate the amount in kg/m^3 of fly ash and ground limestone.

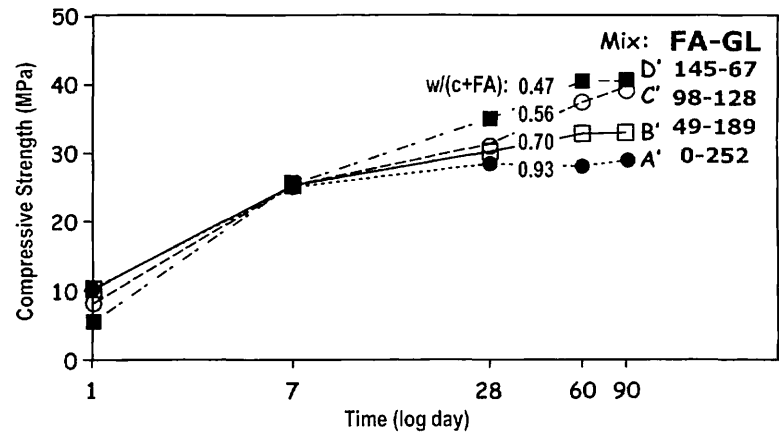


Fig. 7 – Compressive strength as a function of the time for concrete mixtures A', B', C' and D' of Table 1 (superplasticizer: AP). FA and GL indicate the amount in kg/m³ of fly ash and ground limestone.

Several Factors Regarding Sustainability of Marine Concrete Structures

by T. U. Mohammed, H. Hamada, and N. Otsuki

Synopsis:

Several factors regarding sustainability of marine concrete structures are discussed based on detailed laboratory and several long-term exposure tests. The factors include voids at the steel-concrete interface, crack widths and autogeneous healing, blended cements, chemical admixtures, type of bars, and joints. Long-term exposure investigations were carried out utilizing the specimens exposed to the marine environment over 10 years. The following conclusions are drawn:

1. To enhance long-term durability, the steel-concrete interface should be free from voids;
2. Narrower cracks and joint surfaces are healed in the marine tidal environment with alkaline deposits, and thereby control the corrosion of steel bars at these locations;
3. Blended cement with high slag content shows the best performance for long-term durability compared to the normal portland cement and other investigated blended cements;
4. The naphthalene group of chemical admixture shows the best performance for long-term durability compared to other investigated chemical admixtures;
5. Deformed bars are more prone to corrosion compared to plain bars;
6. Brushing-off the laitance and application of a mortar coat at the joint plane is very effective in enhancing long-term durability.

Keywords: blended cements; chemical admixtures; corrosion; joints; steel bars; steel-concrete interface; sustainability; type of bars

24 Mohammed et al.

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INTRODUCTION

Sea covers two-third surface area of the world. The rapid population growth in the world has forced development of more infrastructures at or near the sea, such as wharves, bridges, undersea tunnels, offshore concrete platforms, and the like. Man-made islands to develop airport facilities over the sea is also common in the developed countries due to the lack of free plain land, land acquisition problems, and the like. Moreover, engineers are challenged to develop recreation facilities over the sea by making man-made islands. Also, floating offshore airports in the open sea, floating industries for production of power and oil and gas processing, and floating waste treatment facilities remain a dream of the future. At the same time, it is also realized that the service lifetime of the structures should be lengthened to reduce the environmental problems raised by the concrete industries associated with the energy consumption and carbon dioxide emission in cement production and aggregate processing, destruction of natural quarry sites, and moreover the disposal problems of demolished concrete. The question is how to make marine concrete structures with a long-term service life, such as 100 years or more from the so-called design life of around 30~50 years. A lot of factors are associated with the earlier deterioration of concrete structures from the planning stage, thorough detail design of the structures, detailing of reinforcements, detailing of structural layout, materials selection, construction process, curing, and continuous health monitoring of the structures. Among the earlier deterioration processes, chloride-induced corrosion is recognized as a severe problem all over the world. In this paper, the authors summarize several factors related to the chloride-induced corrosion based on the several laboratory and long-term exposure tests. The factors include voids at the steel-concrete interface, crack widths and autogeneous healing, blended cements,

chemical admixtures, type of bars, and joints. Detail experimental process and results were reported in the referred references and will not be covered here. A summary of these factors will be very useful to the concrete professional concerning the sustainability of marine concrete structures.

SCOPE

The scope of this paper covers several important factors related to the chloride-induced corrosion of steel bars in concrete, such as voids at the steel-concrete interface, crack widths and autogeneous healing, blended cements, chemical admixtures, type of bars, and joints. Detailed experimental processes of the investigations are not provided here, however can be obtained from the referred references. The key results are only summarized here.

VOIDS AT THE STEEL-CONCRETE INTERFACE

This is one of the most important factors related to the corrosion of steel bars in concrete under marine environment. Studies related to corrosion of steel bars in concrete due to the voids at the steel-concrete interface are very scarce in literature (1). The authors carried out two series of detailed laboratory investigations to clarify the process of chloride-induced corrosion with the presence of voids at the steel concrete interface (2-3). Investigation on the concrete specimens after a long-term of exposure in the marine environment was also carried out to compare with the results of laboratory investigations (4). For laboratory investigation, specimens were made with specially fabricated steel bars in concrete, so that the electron generated or absorbed in a particular portion of the bar could be measured. Also, polarization resistance and anodic polarization curves of the particular portions (vertical, top or bottom half of the horizontal steel portions) could be evaluated. This type of detailed investigation is very rare in technical publication. The simplified layout of the experimental setup is shown in Fig. 1. Based on the current flow among the steel elements, the anodic and cathodic areas were separated. The anodic and cathodic areas are marked as 'A' and 'C', respectively (Fig. 1). Generally, the bottom half of the top horizontal steel (HTB) acts as anode due to the formation of voids under the bar caused by the settlement of the unhardened concrete against the fixed steel bar and also the blocking of bleeding water under the bar. The micro-cell current density of the individual steel portions was also estimated from the measured polarization data. A higher micro-cell current density was observed over the bottom portion of the horizontal bar, especially the bottom portion of the top horizontal steel bar (HTB). The value of the micro-cell current density was much higher than the current density defined for a passivity limit at around $0.1 \mu\text{A}/\text{cm}^2$ (5). The anodic polarization

curves of the steel elements were also measured and are schematically represented in Fig. 2. The condition of the passivity over the investigated portions of the steel bar in concrete can be realized from this figure. A complete loss of passivity of the bottom portion of the top horizontal steel bar was concluded based on the corrosion current density (2). Superposition of a significant amount of macro- and micro-cell corrosion over the bottom half of the top horizontal steel (HTB) results in the formation of severe corrosion pits.

In real concrete structures, these voids can be easily created if the casting height is relatively higher, such as under the steel bar located near the top surface of beams, under the stirrups, under the spirals or ties in columns, and under the lug of the deformed bar. It can also around the steel bars placed along the casting direction due to less compaction. Typical corrosion over the bottom part of the top horizontal steel bar of 23-year old beams is shown in Fig. 3. Severe loss of bar diameter was observed after 23 years of exposure. The question is - What will the situation be after 50 or 100 years? Perhaps, complete loss of the steel bar will occur for this case. Therefore, to design long-term durable concrete structures in the marine environment, the steel-concrete interfaces should be free from voids or gaps. Research work is still necessary for this achievement. It is expected that the chloride threshold value may be improved, if it is possible to make a perfect/compact steel-concrete interface without any macro voids or less micro voids. Also, with the presence of voids, corrosion can occur for a very low chloride concentration (4).

It was also observed that if there is a tiny layer of cement paste between the void and the steel bar, the steel bar was not corroded even for a high chloride concentration ($> 2\%$ of cement mass) (6). Therefore, the authors extended their research scope with the pre-passivation steel bars (covering the steel bars with cement paste), re-vibration before final setting, use of expansive cements, and insertion of the steel bars in concrete after casting concrete. The results of this investigation will be presented at a future time.

CRACK WIDTHS AND AUTOGENEOUS HEALING

Generally crack widths are limited in design codes to control corrosion of steel bars in concrete (7-9). A debate supporting the relation between crack widths and corrosion rate is still continuing among the researchers (10-13). The authors have also carried out a detailed laboratory investigation and several long-term exposure tests to confirm the matter. The results of the detailed laboratory investigations were reported in (10). The crack widths were 0.1-0.7 mm. It was realized that there is no significant relationship between the crack widths and corrosion rate except at early age of exposure. To clarify it further, 23-year old concrete beams exposed to the marine atmospheric and tidal environments were investigated. Irrespective of the crack widths, it was

found that rust product was concentrated just at the root of the crack for marine atmospheric pre-cracked beams. In tidal environment, severe loss of bar diameter was observed at the root of the wide crack widths. A typical case is shown in Fig. 4. It seems that the root of the crack over the steel bars acts as anode, with nearby areas acting as cathode to form a macro-cell. Also, the micro-cell current density of the region (marked as anode from the viewpoint of macro-cell corrosion in Fig. 4) will be higher due to the availability of more oxygen through the crack. It is suspected that superposition of higher macro- and micro-cell corrosion over a localized area resulted in the formation of severe corrosion pit around the bar. Such a situation should be avoided in the real concrete structures exposed in the marine environment.

Fortunately, in the marine environment, autogeneous healing can take place, which cannot be observed in the accelerated laboratory tests where 3.5% salt water was generally used. Recently, the authors investigated a large number of 15-year old pre-cracked concrete prism specimens exposed in a tidal pool created by using seawater directly from the sea (14). It was found that the narrow cracks (<0.5 mm) were healed during 15 years of exposure irrespective of the cement types, such as normal portland cement or blended slag cements or blended fly ash cements. Clear white deposits were found in the crack plane after splitting the concrete along the crack. The deposit was even observed at the root of the crack over the steel bars, i.e., the debonded zone at the root of the crack. SEM micrographs over a healed crack are shown in Fig. 5. Deposits of ettringite are clearly observed along with other deposits, such as chloride, sulfate, and magnesium oxide. After spraying a 1% phenolphthalein solution on the split-opened cracked plane, a dark pink color was formed. It indicated that the deposit created a highly alkaline media and thereby stopped the corrosion reaction over the steel bars at the root of the crack. The measured corrosion rates (micro-cell corrosion estimated from the polarization data) at the healed and unhealed cracks are shown in Fig. 6. It is found that for healed cracks, no significant relation between crack widths and corrosion is observed. However, unhealed cracks show a higher corrosion rate. Severe loss of bar diameter at unhealed cracks is shown in Fig. 7, whereas no significant corrosion is found over the steel bars at healed cracks. A clear white deposit is found over the rust formed at the early exposure indicating a repassivation of the bar after some time of exposure. Therefore, the crack widths should be limited to allow possible and quick healing in a marine environment.

Based on the above-mentioned results, it is concluded that the crack widths should be limited to as small of value as possible to allow a healing process to occur. The healing is very essential to ensure long-term durability of concrete structures in a marine environment for periods of 100 years or more. Another important observation is that the process of autogeneous healing was not found over the voids at the steel-concrete interface. Location of the rust and pit, pit size, and distribution of corroded area over a steel bar of 15-year old cracked prism specimen (tidal exposure) are shown in Fig. 8. At the uncracked

regions, due to the voids at the steel-concrete interface several pits are formed. The process of steel corrosion due to the presence of voids at the steel-concrete interface is summarized before. For a marine concrete structure to have a long service life, not only the crack widths but also the elimination of voids at the steel-concrete interface should be taken into account.

BLENDING CEMENTS

The use of blended cements for long-term durability as well as their effectiveness to reduce carbon dioxide emission in the atmosphere is well recognized. Numerous studies on the performance of the blended cements on chloride- and carbonation-induced corrosion have been carried out. The authors also conducted several long-term exposure tests with both cracked prisms (10x10x60 cm) and un-cracked cylinders (15 cm diameter and 30 cm length) made with normal portland cement and blended cements, such as slag cement Types A, B and C (SCA, SCB and SCC) and fly ash cement Type B (FACB). The specimens were exposed in a tidal pool created by using seawater directly from the sea to simulate the real marine tidal environment. The results of the investigation on the performance of blended cements in cracked and un-cracked concrete specimens are reported in (14) and (15). It was found that the concrete made with normal portland cement has less concrete resistance and polarization resistance (more current density as the corrosion rate is inversely proportional to the polarization resistance) over the steel bar than for concrete made with blended cements. The water- and acid-soluble chloride profiles of the investigated cements are shown in Fig. 9. In SCA, SCB and SCC slag contents are 5~30, 30~60 and 60~70%, respectively. In FACB, fly ash content is 10~20%. It was found that for concrete with blended slag cements, chloride concentration quickly drops to a negligible value within a short distance inside the specimens, especially for SCC. Based on the chloride profile, the apparent diffusion co-efficient was estimated and listed in Table 1. The time to initiate corrosion was also estimated based on the chloride threshold limit at 0.4% of cement mass (16). Estimated time to initiate corrosion is also listed in Table 1.

Slag cement, with a high amount of slag (SCC), shows the best performance to lengthen the time to initiate corrosion. The time necessary to reach a chloride level of 0.4% of cement mass over the steel bars located at 70 mm of cover is estimated at 150 years for SCC. It is recognized that blended slag cement is very effective to enhance long-term durability of marine concrete structures. The pre-cracked specimens made with different kinds of cement were also investigated. As for both uncracked concrete and concrete with narrower crack widths, the blended cements with a higher amount of slag content (SCC) showed the best performance to control corrosion over the bars. However, for wider crack widths (unhealed cracks), the use of blended cements was ineffective. More explanation on this matter is provided later.

Compressive strength data of 15-year old concrete specimens were reported in (15). No reduction in strength was observed, especially for normal portland and slag cements. Relatively higher gain in strength was found in concrete specimens made with slag cements.

To explain the chloride accumulation at the surface region of the specimens made with blended cements, detailed investigations, such as porosity, XRD, SEM evaluation of interfaces and matrix in concrete were carried out. The results were reported in (17-18). The porosity data at the outer and inner regions of the specimens made with slag cement (60~70% replacement) are shown in Fig. 10. For slag cement, the capillary pore volume at the outer region of the specimens was significantly reduced compared to the inner region of the specimen. The denser microstructure of blended cements screens the diffused ions at the outer region and thereby improves the microstructure further (Fig. 10). Unfortunately, for wider crack widths (unhealed cases), the performance of blended cements seems doubtful due to the accumulation of chlorides at the un-healed cracked regions. Also, the difference of chloride concentration over the steel bar at the cracked region and the uncracked region is much higher for slag cement, as less amount of chloride is observed over the steel bars at the uncracked region. This causes an acceleration of the corrosion rate and results in severe loss of bar diameter.

CHEMICAL ADMIXTURES

Several chemical admixtures were developed mainly to improve the performance of concrete at the early age, reduce shrinkage at the early age, reduce the water demand, and the like. However, reports on the long-term performance of these chemical admixtures from the durability viewpoint are very scarce in the technical literature. Recently, the authors carried out an investigation using 10-year-old plain and reinforced concrete cylinder specimens (diameter 10 cm and length 20 cm) made with different chemical admixtures. Chemical admixtures included air-entraining admixture (vinsol), water-reducing admixture (lingosulfonate group), various high-range water-reducing and air-entraining admixture (naphthalene, melamine, polycarboxyl and amino-sulfonate group) and drying shrinkage reducing admixture (glycol ether plus amino alcohol derivatives). The specimens were exposed in the splash and tidal environments. The detail results were reported in (6 and 19). The chloride profiles are shown in Fig. 11 for the specimens made with different high-range water-reducing and air-entraining chemical admixtures after 10 years of exposure in the tidal environment. Naphthalene, melamine, polycarboxyl and amino-sulfonate groups of chemical admixtures are denoted as N, M, PC and AS, respectively. It was found that naphthalene group of chemical admixture shows relatively better performance with respect to the chloride ingress. Relatively, more chlorides were penetrated at the inner region

of the specimens made with polycarboxyl group of chemical admixture. Also, a reduction in compressive strength was observed for the polycarboxyl group of chemical admixture (19). The exact reason cannot be concluded at this stage. Further long-term exposure investigations are suggested to confirm the long-term performance of polycarboxyl group of water reducing chemical admixture. Another series of the specimens exposed to the marine splash zone was also investigated and reported in (20). Almost the same results as in the tidal exposure were observed. No long-term harmful effect of using the investigated shrinkage reducing chemical admixture was found from these investigations.

PLAIN AND DEFORMED BARS

Deformed bars are commonly used for their good bonding ability with concrete. However, no detail investigation on the relative durability of plain and deformed bars could be identified. The authors carried out a comparative study on the corrosion of plain and deformed steel bars in cracked concrete. The results were reported in (10 and 21). A view of the exposure site (size of the beam 15x15x125 cm) is shown in Fig. 12. Saltwater (3.5%) was sprayed once a week over the beams. The polarization resistance and concrete resistance over the bar in cracked concrete beams are shown in Fig. 13. Due to the extensive micro-cracking in cracked concrete beams made with deformed bars as well as more surface cracks, the concrete resistance reduced significantly. Higher chloride concentration over the deformed bar was also observed for this reason (10). To clarify the higher corrosion rate over the deformed bar, further investigation on the oxygen permeability was also carried out. The results were reported in (21). It was found that cracked concrete made with deformed bar show more oxygen permeability compared to the cracked concrete made with plain bar. After 16 months of exposure, steel bars were collected from the several beams to see the rust on the bar. More corrosion and deeper corrosion pits were observed on the deformed bars (10).

JOINTS

A joint is a surface where two successive placements of concrete meet, across which it may be desirable to achieve bond and through which reinforcement may be continuous. The occurrence of joints cannot be avoided for many reasons, such as changing in lift, continuous casting, and the like. The authors carried out a detailed experimental investigation on the performance of construction joint preparation under marine atmospheric and tidal environment. The specimens (diameter 15 cm and length 30 cm with joint at the middle) were investigated after 25 years of exposure. The investigated joints are defined in Table 2. The results related to the performance of joints under

marine tidal environment were reported in (22). The results related to the marine atmospheric environment are reported in this conference separately (23). Under the tidal environment, autogeneous healing controls the corrosion of steel bars at the joint region irrespective of the type of joint treatment. The presence of ettringite was clearly observed over the joint surface as shown in Fig. 13. Irrespective of the joint treatment, the healing process was observed. For a joint type made with epoxy coating, corrosion was observed over the steel bars at the joints due to the unexpected formation of voids caused by the shrinkage of the epoxy. Also, corrosion over the steel bars was found at the locations far from the joints due to the voids at the steel-concrete interface. Macro- and micro-cell corrosion were also measured after 25 years of exposure. The steel bar at the joint region did not necessarily act as an anode, also the micro-cell corrosion current density was not necessarily higher over the steel bars at the joint region (22).

In an atmospheric environment, the carbonation front reached over the steel bars through the joints and extended through the steel-concrete interface beside the joint. For these specimens, macro- and micro-cell corrosion were measured same as tidal specimens. Generally, the steel bars at the joint region act as an anode with a higher degree of micro-cell corrosion observed at the joint region. This is because of the movement of the carbonation front through the joint. Based on the electro-chemical and physical investigations, it was concluded that the removal of the laitance from the joint surface is very important to limit the carbonation front. With respect to healing in the tidal environment, the removal of the laitance showed no influence. For long-term durability against corrosion of steel bars in concrete, application of a mortar coating after removal of the laitance was found to be very effective.

In the earlier reported references, the quantitative evaluation of the strength of the joints was not reported as no specimens were made without reinforcements. Recently, an attempt has been taken to quantitatively evaluate the tensile bond strength at the joints of the tidal specimens, after 25 years of exposure. For this, 60 mm cube specimens were cut from the specimens keeping the joints at the middle. Then the bond strength was evaluated after fastening plates at the ends. The bond strengths are listed in Table 2. The removal of the laitance from the joint plane is necessary to improve the bond strength. Brushing-off the laitance and application of an epoxy coating shows the highest bond strength. Interestingly, the higher bond strength does not result in less corrosion in the tidal or atmospheric environment.

CONCLUSIONS

Based on the several detailed laboratory and long-term exposure tests summarized here, the following conclusions are drawn:

1. To enhance long-term durability, steel-concrete interface should be free from voids.
2. Narrower cracks heal under the tidal environment and stops corrosion over steel bars at the root of the crack. Deep localized corrosion is observed for wider, unhealed cracks. Also, deep pits were observed at the uncracked region with voids at the steel-concrete interface.
3. Autogeneous healing/deposits at the narrower cracks/joints reduce the corrosion rate significantly under the marine tidal environment.
4. Blended cement with high slag contents shows the best performance for long-term durability compared to normal portland cement and other blended cements.
5. Naphthalene group of chemical admixture shows the best performance for long-term durability compared to the other investigated chemical admixtures.
6. Deformed bars are more prone to corrosion compared to plain bars due to the more internal and surface cracks in cracked concrete made with deformed bars.
7. Brushing-off the laitance and application of a mortar coating is very effective in reducing the corrosion rate at the joint compared to the use of epoxy or latex paste coats. Brushing-off the laitance and application of an epoxy coating shows the strongest bond strength, but it did not show the best performance in reducing corrosion of the steel bar.

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34 Mohammed et al.

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Table 1. Diffusion Coefficient and Time to Initiate Corrosion

Specimen	Diffusion Coefficient (mm ² /year)	Time to Initiate Corrosion (Years)		
		2 cm	4 cm	7 cm
NPC	75	2	7	22
SCA	16	6	26	79
SCB	12	8	32	98
SCC	8	12	49	150
FACB	15	5	21	65

NPC – Normal Portland Cement, SCA – Slag Cement A

SCB – Slag Cement B, SCC – Slag Cement C.

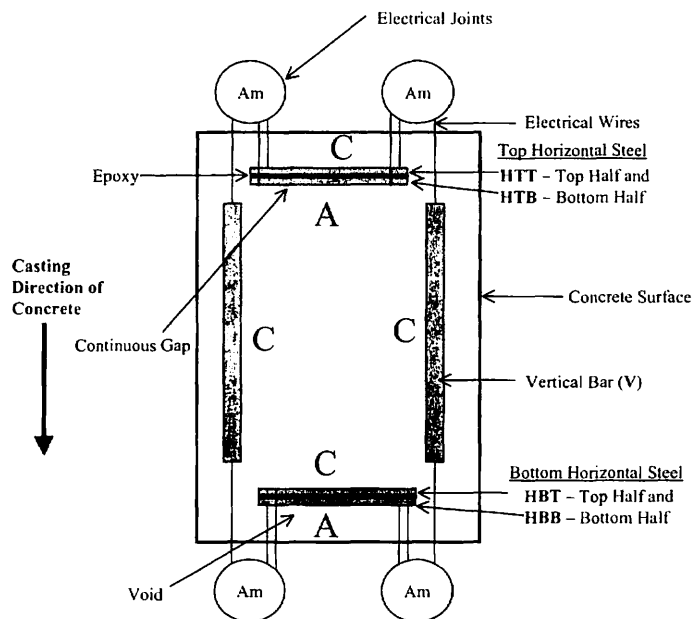
W/C=0.45, Tidal Exposure for 15 Years.

Table 2. Tensile Bond Strength of Joints after 25 Years of Tidal Exposure

Type	Treatment Applied	Bond Strength (MPa)
A	No treatment for the laitance	-
B	No treatment for the laitance but place a 5-mm-thick mortar coating	-
C	Brushing-off the laitance	2.3
D	Brushing-off the laitance and place a 5-mm-thick mortar coating	1.3
E	No treatment for the laitance and place an epoxy coating	1.2
F	Brushing-off the laitance and place an epoxy coating	4.1 *
G	Brushing-off the laitance and place a latex paste coating	0.8

- Broken during sampling due to the very weak bond strength

* not broken at the joint.



Am = Zero Resistance Ammeter
A = Anodic Area, C = Cathodic Area

Fig. 1 Simplified Cross-Section of the Specimens Used for Detailed Laboratory Investigation and Locations of Anodic (A) and Cathodic (C) Areas

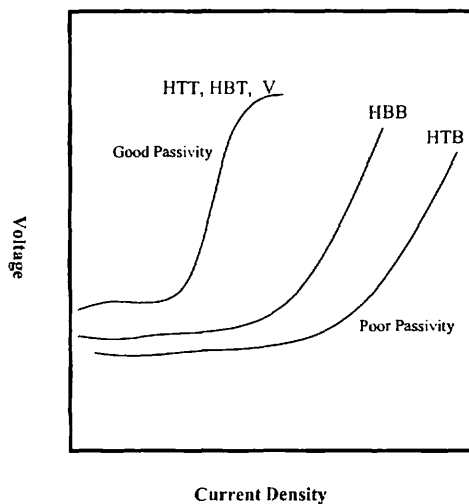
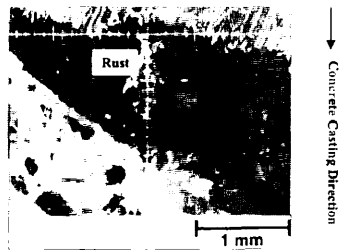


Fig. 2 Schematic Diagram of Anodic Polarization Curves

W/C=0.7, 23 Years Tidal Exposure
Water Soluble Chloride Concentration = 1% of Cement Mass



W/C=0.7, 23 Years Tidal Exposure.
Water Soluble Chloride Concentration = 2% of Cement Mass

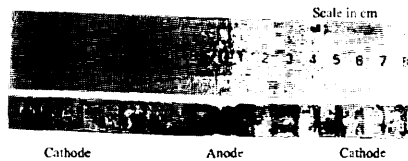


Fig. 3 Corrosion Under Top Horizontal Bar (HTB of Fig. 1)

Fig. 4 Severe Loss of Bar Diameter at a Wide Crack

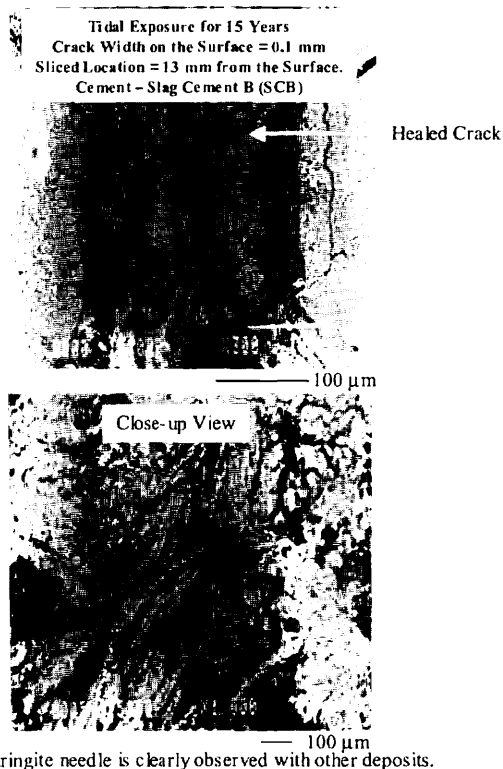


Fig. 5 SEM Micrographs – Deposit at the Crack

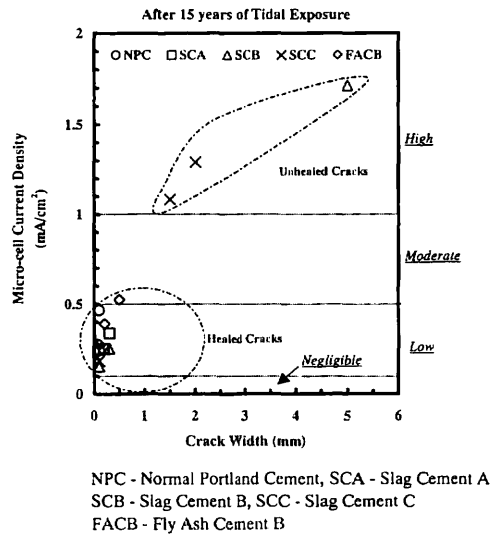


Fig. 6 Macro-Cell Corrosion of Steel Bars the at Different Cracks

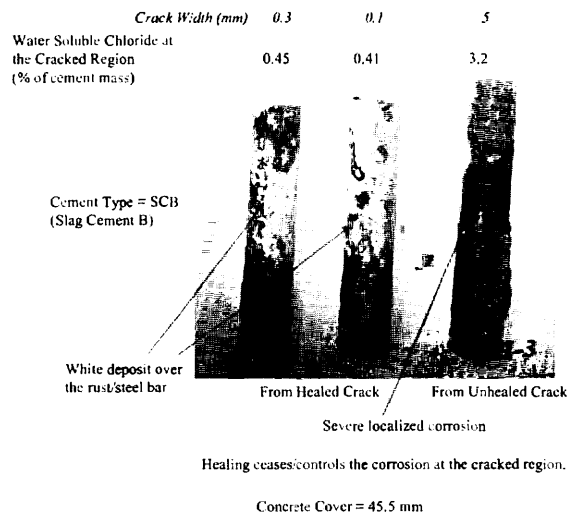


Fig. 7 Corrosion at the Healed and Unhealed Cracks of 15 Years Old RC Beams Exposed to the Tidal Environment

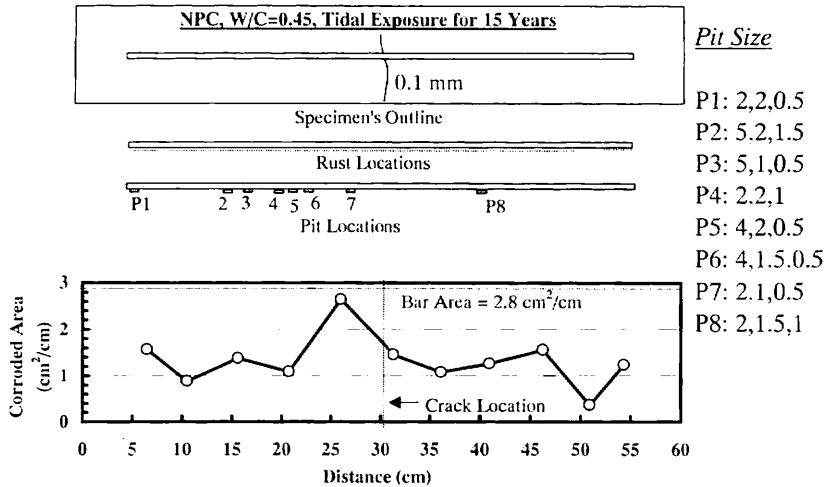


Fig. 8 Rust Locations, Pit Locations and Size, and Distribution of Corroded Area

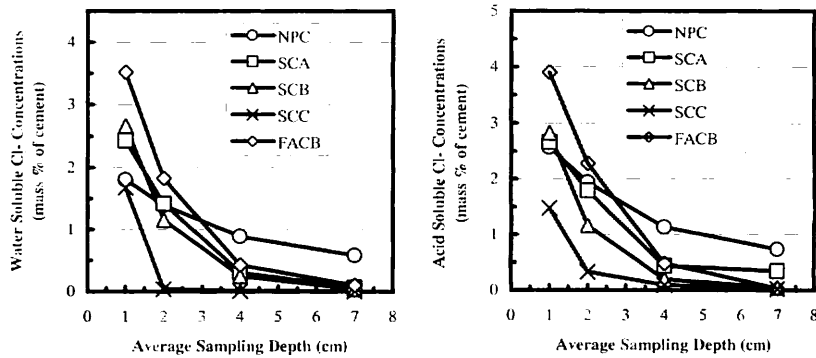


Fig. 9 Water- and Acid-Soluble Chloride Profiles in Concrete

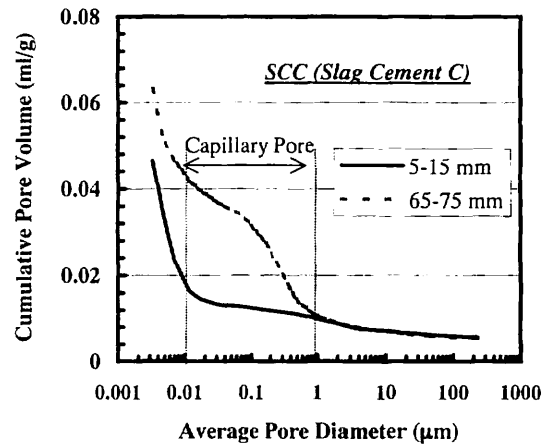


Fig. 10 Porosity at the Inner and Outer Regions of the Specimens Made with Slag Cement C

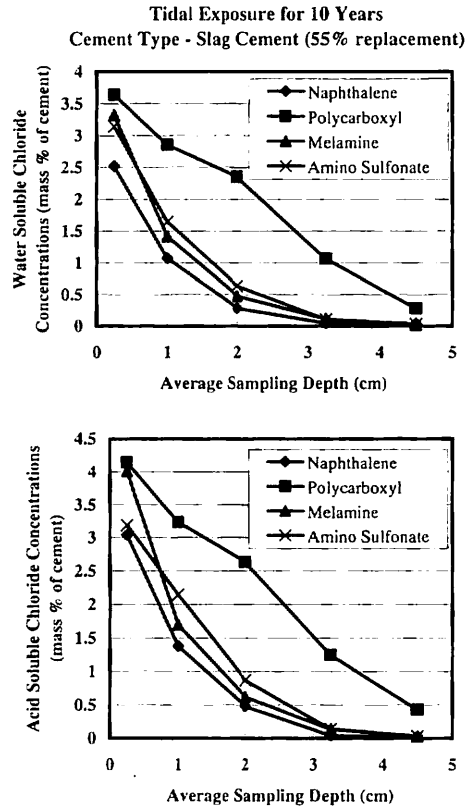
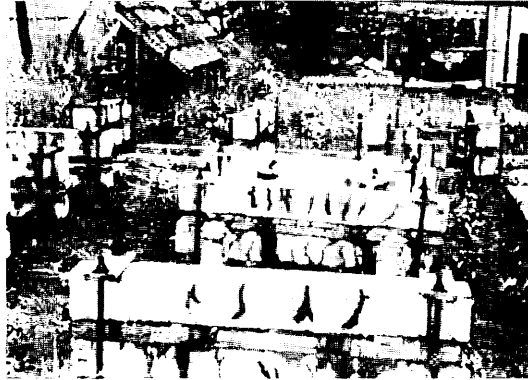


Fig. 11 Water- and Acid-Soluble Chloride Profiles in Concrete Made with Different Chemical Admixtures



Specimen Size 125 (length) $\times$ 15 (width) $\times$ 15 (depth) cm

Concrete Cover = 2.5 cm

The photograph is taken sometime after spraying salt water.

Beams with plain bars show less number of cracks.

Fig. 12 Partial View of the Exposure Site

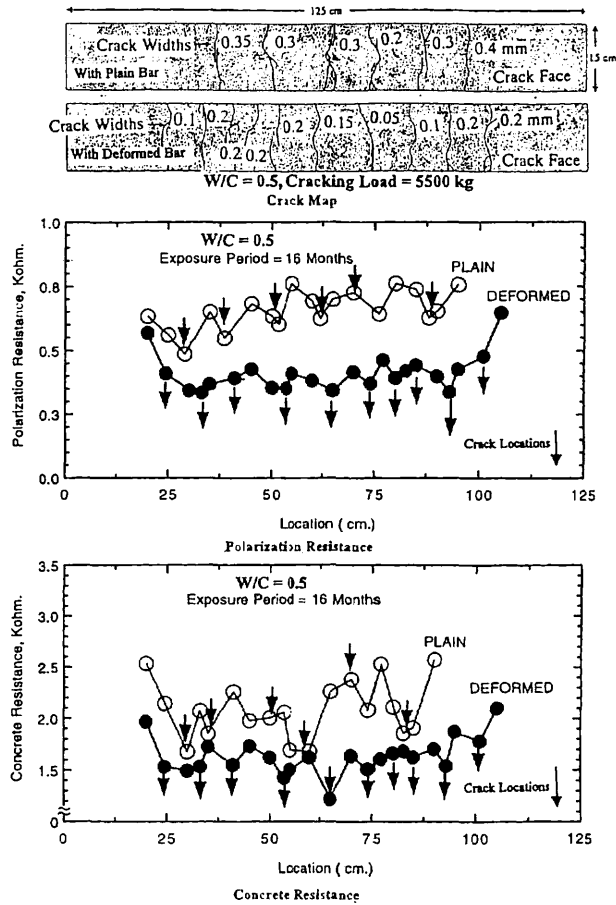


Fig. 13 Cracked Face of the Beam with the Distribution of Polarization Resistance and Concrete Resistance along the Beam



Fig. 14 SEM Micrograph Showing the Deposit at the Joint

Long-Term Durability of Surface Protection Systems

by M. Raupach and L. Wolff

Synopsis:

Polymeric surface protection systems can be a durable and potent method to lower the moisture content of the concrete and to improve its durability by inhibiting the penetration of aggressive substances through the surface as well as by limiting the corrosion rates.

At the Institute for Building Materials Research in Aachen, ibac, in cooperation with BAM in Berlin comparative tests have been carried out on the long-term durability of selected classes of surface protection systems financed by the German Ministry of Transport. These systems were weathered naturally at two different locations in Germany as well as artificially in different climatic test facilities. Afterwards the mass and dimension changes of the polymeric films, adhesion strength, diffusion of chlorides and carbon dioxide, cross-cut and crack bridging behaviour have been investigated.

Keywords: concrete; durability; surface protection system



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46 Raupach and Wolff

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INTRODUCTION

This paper presents investigations on selected classes of surface protection systems for concrete. These systems were artificially weathered in a climatic test facility as well as naturally weathered over five years at different locations in Germany. These locations were the village Westerland on the Northern Sea island Sylt and the city Duisburg in the industrial area of the river Ruhr. Westerland has a very humid climate with chlorides while Duisburg represents an industrial climate with medium air pollution.

Today the main verification parameters for the durability of surface protection systems are adhesion strength and crack bridging properties. In elder guidelines the elasticity and elongation at break of the polymeric films were also tested. On the basis of the required tests according to the guidelines different types of specimens were investigated. All in all 18 different surface protection systems were part of the research programme realised by the two participating institutes Federal Institute for Materials Testing BAM in Berlin as well as the Institute for Building Materials Research of the Technical University Aachen.

The systems belong to six different classes of surface protection systems according to /1/ and /2/. Besides a hydrophobic treatment all systems are suited for not-trafficable faces and facades. Three systems have certain additionally crack bridging properties.

Only the results of two classes of a crack-bridging surface protection system are represented in this paper as well as the different tests carried out at ibac. The total testing program is including all results and a discussion is available in German /4/.

BASIC GUIDELINES FOR SURFACE PROTECTION SYSTEMS IN GERMANY

In Germany surface protection systems are regulated in two different guidelines: The guideline “protection and repair of concrete structures” covers all types of concrete structures and has been published by the German Committee for Reinforced Concrete DAfStb /1/. This regulation was published in a revised form in October 2001. The other corpus of legislation “Additional technical terms of contract for protection and repair of concrete structures ZTV SIB” is published by the German Ministry for Transport BMV /2/ covering mainly the field of bridges. The presented surface protection systems all passed the basic tests according to the second named guideline.

TEST SPECIMENS

On the basis of the required tests according to the named guideline ZTV SIB selected types of specimens were produced. In Fig. 1 the specimens for testing the crack-bridging properties as well as the test set-up for measuring the adhesion strength are represented. In Fig. 2 the different types of construction for the natural weathering in Duisburg and Sylt are shown.

MATERIALS

Six different hydrophobic agents, impregnations and surface protection systems were investigated under the presented research programme. In each class three different products of different producers were selected. The properties of the different systems are stated below. The different film thickness which is stated in the description of each system is mainly necessary for crack bridging properties of the coating as well as to ensure the required diffusion coefficient for carbon dioxide.

- Hydrophobic agents based on different chemical compositions of silanes, OS 1 (OS A) (/1/,/2/)
- Dispersion coating for not trafficable faces, as a preventative protection, medium film thickness about 80 µm, OS 2 (OS B) (/1/,/2/)
- Dispersion coating for not trafficable faces, facades or side faces of bridges as part of a repair measure, medium film thickness about 80 µm, additional base coat (fine spackle), OS 4 (OS C), (/1/,/2/)

48 Raupach and Wolff

- Dispersion coating for not trafficable faces, facades or side faces of bridges as part of a repair measure, medium film thickness about 300 μm , fine spackle and additional base coat, crack bridging properties to a maximum crack width of 0,15 mm (dynamic $\Delta c=0,05$ mm), OS 5a (OS DII) (/1/,/2/)
- Polymer cement mixture for not trafficable faces, facades or side faces of bridges as part of a repair measure, medium coating thickness about 2 mm, additional base coat (fine spackle), crack bridging properties to a maximum crack width of 0,15 mm (dynamic $\Delta c=0,05$ mm), OS 5b (OS DI) (/1/,/2/)
- polymeric coating for not trafficable faces, facades or side faces of bridges as part of a repair measure, medium film thickness about 1000 μm , fine spackle and additional base coat, crack bridging properties to a maximum crack width of 0.3 mm (dynamic $\Delta c=0,2$ mm), OS 9 (OS E) (/1/,/2/)

The first three surface protection systems were investigated at the Federal Institute for Materials Testing BAM in Berlin and the further systems were investigated at the Institute for Building Materials Research in Aachen.

GENERAL

The different tests carried out at ibac are presented in table 1. Additional tests about the diffusion coefficient of carbon dioxide and chlorides have been carried out at BAM. As reference all tests have been carried out before weathering. Additional tests have been carried out after the different steps of artificial weathering as well as one year, three years and five years of natural weathering. Next appraisal will be done 10 years after production of the specimens.

ARTIFICIAL WEATHERING

Two different types of artificial weathering were selected. The second type (AW 2) as well as the freezing-thawing cycles (FTC) are required by the ZTV SIB /2/. The detailed weathering cycles are represented in table 2.

NATURAL WEATHERING

The natural weathering was done at the named locations Duisburg and Sylt. The climatic conditions as temperature, humidity, times and duration of rainfall were recorded by the local weather stations. In the observed years Duisburg showed higher medium temperatures over the whole year with similar freezing-thawing cycles compared to Sylt. The highest medium air humidity was measured in Sylt but less scores of rainfall. The global radiation was about equal between the two locations.

RESULTS AFTER FIVE YEARS

General

In this paper only the results of the class DI and DII surface protection systems after different steps of artificial and natural weathering are represented. The class DI system is common in Germany for the maintenance and repair of constructions made of concrete like water gates or constructions for the inland navigation. The class DII products are often used for the sides or bottom side of bridges or prestressed beams.

Mass and dimension changes of the polymeric films

The mass changes of the films were tested by weighing before and after different steps of artificial and natural weathering. The dimension changes were measured by determination of the distance between different marks on the backside of the specimen. The results of change of mass and change of dimensions are represented in figures 3 and 4.

The results show that all products loose between 10 and 20 M.-% after three years of exposure. Further tests showed that this loss of mass occurs already in the first months of natural exposure. The change of loss of mass between three and five years is in most cases negligible. The difference between Sylt and Duisburg is not in all cases consistent. In some cases is the loss of mass of the specimens weathered in Duisburg a little bit higher than of the specimens weathered in Sylt.

The medium film thickness does not show any tendency neither according to the system nor to the location. The measured values vary between +30 and – 25 %. The changes of mass seem to be based on swell and shrink dependant on the scores of rainfall as well as the medium air moisture.

50 Raupach and Wolff

Elongation at break of the polymeric films

According to /2/ the elongation at break was measured at +23 °C and –20 °C respectively. The results of the tensile test of the polymeric films at the temperature of –20 °C are shown in figures 5 and 6. The dimensions of the test specimen were 10 x 200 mm² with a film thickness as applied on the test specimens as shown in fig. 2. The test was done with a test speed of 1 mm/min up to an elongation of 2 %. After elongation of 2 % the test was done with a test speed of 10 mm/min at a temperature of –20 °C and 100 mm/min at a temperature of 23 °C. The shaded areas in the figures represents the elongation at break after artificial weathering cycles AW 1 and AW 2.

All investigated polymeric films showed a decreasing elongation at break both after artificial and natural weathering. In some cases it was no longer possible to test the exposed film by means of destruction by wind or hail. The reduction of elongation at break is much larger at the Class DI systems which are polymer cement mixtures. The polymeric films of the DII systems showed only a decrease of elongation at break after 5 years of about 25 to 60 % while the polymer cement mixtures lost about 60 to 90 percent according to the reference.

Crack bridging properties

The specimen used for measuring the crack bridging properties have already been shown in figure 1. The regulated test procedure according to /1/ or /2/ is by testing a defined crack bridging class, which contains a minimum and maximum crack width, a defined frequency for testing the dynamic crack bridging properties as well as a number of cycles to be tested.

To evaluate the maximum crack bridging properties of the mortar specimens and to compare the results of the mortar specimens with the elongation at break of the free films one of the mortar specimens of each class was tested by expanding the crack in the mortar until the surface protection system showed indications of a failure like constriction or holes. The temperature under the test was constantly –20 °C. The results of the maximum crack bridging properties of the class DI and class DII systems are represented in fig. 7 and 8.

The maximum reduction in crack bridging properties is about 50 % which means a significant lower reduction in contrast to the free polymeric films. While all free polymeric films showed decreasing elongation at break the maximum crack bridging crack width of the mortar specimen increased over time in both cases of the DI system and two of three cases of the DII system.