

Service life prediction of a reinforced concrete bridge exposed to chloride induced deterioration

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Abstract. While recognizing the problem of reinforcement corrosion and premature structural deterioration of reinforced concrete (RC) structures as a combined effect of mechanical and environmental actions (carbonation, ingress of chlorides), emphasis is given on the effect of the latter, as most severe and unpredictable action. In this study, a simulation tool, based on proven predictive models utilizing principles of chemical and material engineering, for the estimation of concrete service life is applied on an existing reinforced concrete bridge (Øresund Link) located in a chloride environment. After a brief introduction to the structure of the models used, emphasis is given on the physicochemical processes in concrete leading to chloride induced corrosion of the embedded reinforcement. By taking under consideration the concrete, structural and environmental properties of the bridge investigated, an accurate prediction of its service life is taking place. It was observed that the proposed, and already used, relationship of service lifetime- cover is almost identical with a mean line between the lines derived from the minimum and maximum critical values considered for corrosion initiation. Thus, an excellent agreement with the project specifications is observed despite the different ways used to approach the problem. Furthermore, different scenarios of concrete cover failure, in the case when a coating is utilized, and extreme deicing salts attack are also investigated.

Keywords: bridge; chloride attack; concrete; durability; failure scenarios; protective measures; service life

1. Introduction

The majority of concrete deterioration cases is connected to corrosion of reinforcement due to carbonation- or chloride-induced depassivation of steel bars. In urban and industrial areas, where environmental pollution results in a significant concentration of carbon dioxide, carbonation-initiated reinforcement corrosion prevails. Numerous surveys have indicated that chloride ions, originating from deicing salts or seawater, are the primary cause of reinforcing steel corrosion in highways and marine or coastal structures (Shi *et al.* 2012). The chlorides that transported through the concrete pore network and microcracks depassivate the oxide film covering the reinforcing steel and accelerate further the reaction of corrosion. Even high-performance concrete may not necessarily ensure long-term durability in a severe environment unless it is designed for dimensional stability and soundness.

Over the past 50 years, an enormous amount of energy has been expended in laboratory and

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field studies of chloride effect on concrete durability. Despite this effort, it is still not possible to identify the “ideal” concrete to provide optimum performance in a particular corrosive environment, because of the numerous material, design and environmental parameters involving into this problem (Mehta and Monteiro 2006). A fundamental approach is, therefore, absolutely necessary.

In the present work, a simulation tool, based on proven predictive models utilizing principles of chemical and material engineering, for the estimation of concrete service life (according to in performance-related methods of the European Standard EN 206 for assessing durability) is applied on an existing reinforced concrete bridge (Øresund Link; connecting Denmark and Sweden) located in a chloride-rich environment. After a brief introduction to the structure of the models used, emphasis is given on the physicochemical processes in concrete leading to chloride induced corrosion of the embedded reinforcement. By taking under consideration the concrete compositional parameters, the environmental conditions and various structural properties of the bridge investigated, an accurate prediction of its service life is taking place. Furthermore, different scenarios of concrete cover failure, in the case when a coating is utilized, and extreme deicing salts attack are also investigated. Overall various protective measures are suggested for secure protection against chloride attack.

2. Mathematical model

Almost in the majority of papers, chloride transport in concrete is modeled using the Fick's second law of diffusion neglecting the chloride interaction with the solid phase. However, several field studies in recent years have indicated that the strict use of this law is not applicable for long term chloride transport into concrete, calculating very often a decreasing chloride transport coefficient in time (Tang *et al.* 2012). It is widely accepted that the transport behavior of chloride ions in concrete is a more complex and complicated transport process than what can be described by Fick's law of diffusion (Glasser *et al.* 2008, Zhang and Gjrv 1996, Tang *et al.* 2012). This approach, therefore, can be characterized as empirical, resulting in the calculation of an “apparent” effective diffusivity unable often to explain chloride penetration and confrontation practices.

There is a generally good correlation between C_3A -content (or C_4AF when there is lack of C_3A -phase) and chloride binding capacity. There is also evidence for binding of chlorides in CSH gel, possibly in interlayer spaces (Beaudoin *et al.* 1990). The Na^+ ions can be bound in CSH gel lattice (Suryavanshi *et al.* 1996), especially when the C/S ratio is low. Several secondary chloride-calcium compounds have also been reported (Roy *et al.* 2000). In addition to the chemical binding, the effects of ionic interaction, lagging motion of cations and formation of electrical double layer on the solid surface are all playing an important role in the transport of chloride ions in concrete (Zhang and Gjrv 1996).

The relationship between bound and free chlorides is non-linear and may be expressed by Langmuir equation (Pereira and Hegedus 1984), Freundlich equation or modified BET equation (Tang *et al.* 2012). Among them the Langmuir equation is both fundamental and easier for practical applications. Pereira and Hegedus (1984) were the first to identify and model chloride diffusion and reaction in fully saturated concrete as a Langmuirian equilibrium process coupled with Fickian diffusion. Furthermore, Papadakis *et al.* (1996) and Papadakis (2000) generalized this pioneering model effort of Pereira & Hegedus and extended to more general conditions, offering an alternative simpler, yet equally accurate, numerical and analytical solution. By introducing a

chloride-solid phase interaction term in the equations the calculation of an “intrinsic” effective diffusivity is possible.

The physicochemical processes of diffusion of Cl^- in the aqueous phase of the concrete pores, their adsorption and binding in the solid phase of concrete, and their desorption therefrom are described by a nonlinear partial differential equation for the concentration of Cl^- in the aqueous phase $[\text{Cl}^-(aq)]$ (in kg/m^3 pore solution), from which that of Cl^- bound in the solid phase $[\text{Cl}^-(s)]$ (kg/m^3 concrete) can be computed algebraically (Papadakis 2000)

$$\frac{\partial [\text{Cl}^-(aq)]}{\partial t} = \frac{D_{e,\text{Cl}^-} (1 + K_{eq} [\text{Cl}^-(aq)])^2}{K_{eq} [\text{Cl}^-(s)]_{sat} + \varepsilon (1 + K_{eq} [\text{Cl}^-(aq)])^2} \frac{\partial^2 [\text{Cl}^-(aq)]}{\partial x^2} \quad (1)$$

$$[\text{Cl}^-(s)] = \frac{K_{eq} [\text{Cl}^-(aq)]}{1 + K_{eq} [\text{Cl}^-(aq)]} [\text{Cl}^-(s)]_{sat} \quad (2)$$

$$\text{initial condition: } [\text{Cl}^-(aq)] = [\text{Cl}^-(aq)]_I \quad \text{at } t = 0 \text{ (initial concentration)} \quad (3)$$

$$\text{boundary conditions: } [\text{Cl}^-(aq)] = [\text{Cl}^-(aq)]_0 \quad \text{at } x = 0 \text{ (concrete surface)} \quad (4a)$$

$$\partial [\text{Cl}^-(aq)] / \partial x = 0 \quad \text{at } x = M \text{ (axis of symmetry)} \quad (4b)$$

In these equations, x is the distance from the concrete surface (m), t is the time (s), D_{e,Cl^-} denotes the intrinsic effective diffusivity of Cl^- in concrete (m^2/s), K_{eq} the equilibrium constant for Cl^- binding (m^3 of water pore volume/kg), $[\text{Cl}^-(s)]_{sat}$ the saturation concentration of Cl^- in the solid phase (kg/m^3 concrete), and ε the concrete porosity (m^3 pore volume/ m^3 concrete). As observed from Eq. (2), the chloride binding capacity depends both on $[\text{Cl}^-(s)]_{sat}$ (content of sites which can bind chlorides) and K_{eq} (ratio of adsorption to desorption rate constants). Eq. (1) can be solved only numerically, e.g., using a finite difference or element method. The solution allows estimation of the time (critical time for chloride-induced corrosion, t_{cr}) required for the chloride concentration surrounding the reinforcement to increase over the threshold of depassivation of reinforcing bars, $[\text{Cl}^-(aq)]_{cr}$.

As observed from Eq. (1), chloride ingress is retarded as D_{e,Cl^-} decreases, $[\text{Cl}^-(s)]_{sat}$ increases, or K_{eq} increases. When a *supplementary cementing material (SCM)*, such as silica fume (SF) or fly ash (FA), is added in concrete, the chloride binding capacity increases, as experimental results showed (higher total chloride content in a thin slice near the external concrete surface; Papadakis 2000, Papadakis *et al.* 2007). This may be attributed to higher CSH content, especially that with lower C/S ratio, which can bind Na^+ ions and, therefore, the accompanied Cl^- . On the other hand, the pore restructuring due to pozzolanic products may decrease intrinsic diffusivity as well. As observed (Papadakis 2000), a fine network of pozzolanic product (CSH) is created in the middle of a capillary pore acting as trap for chlorides. Using atomic force microscopy (Papadakis *et al.* 1999), it was obtained that the internal surface of the SCM-cement pastes presents small spheroid bulges giving an additional pore roughness.

2.1 Parameter estimation

Let us suppose that 1 m^3 of fresh concrete that contains potentially both silica fume and fly ash is composed as follows:

C : kg cement / m ³	ρ_C : cement density (kg/m ³)
SF : kg silica fume / m ³	ρ_{SF} : silica fume density (kg/m ³)
FA : kg fly ash / m ³	ρ_{FA} : fly ash density (kg/m ³)
A : kg aggregates / m ³	ρ_A : aggregate density (kg/m ³)
W : kg water / m ³	ρ_W : water density (kg/m ³)
D : kg admixtures / m ³	ρ_D : admixture density (kg/m ³)
ε_{air} : m ³ of entrained or entrapped air / m ³	

The following balance equation should be fulfilled:

$$C/\rho_C + SF/\rho_{SF} + FA/\rho_{FA} + A/\rho_A + W/\rho_W + D/\rho_D + \varepsilon_{air} = 1 \quad (5)$$

It has been shown (Papadakis 1999a, b), that if α is the portland cement hydration degree, and β the fly ash reaction degree, the total porosity of a non-carbonated concrete, ε is given as follows (SF does not practically influence the total porosity)

$$\varepsilon = \varepsilon_{air} + W/\rho_W - 0.227 \cdot 10^{-3} \alpha C - 0.188 \cdot 10^{-3} \beta FA \quad (6)$$

The intrinsic effective diffusivity of Cl⁻ in concrete (m²/s) can be estimated by the following semi-empirical equation (Papadakis 2000)

$$D_{e,Cl^-} = \frac{2.4 \cdot 10^{-10}}{\left(\frac{C + k_{SF} SF + k_{FA} FA}{\rho_C} + \frac{W}{\rho_W} \right)^2} (\varepsilon_{eff})^3 \quad (7)$$

Eq. (7) is valid for NaCl and for $0.4 < W/C < 0.7$, $0 < A/C < 6$; for CaCl₂ the numerator is 2×10^{-10} . For $W/C < 0.5$, the dependence of D_{e,Cl^-} from ε_{eff} is slightly higher, i.e., $(\varepsilon_{eff})^{3.5}$. In Eq. (7) k_{SF} and k_{FA} are the efficiency factors (*k-values*) of silica fume and fly ash respectively regarding chloride penetration, and ε_{eff} an effective, for diffusion, porosity, calculated as follows

$$\varepsilon_{eff} = W/\rho_W - 0.227 \cdot 10^{-3} \alpha (C + k_{SF} SF + k_{FA} FA) \quad (7a)$$

If sea attack concerns, the chloride concentration in the aqueous solution at the concrete surface, $[Cl(aq)]_0$ (kg/m³ pore solution), depends on the sea, e.g., Atlantic ocean: 20 kg/m³, North Sea: 16 kg/m³, Baltic Sea: 4 kg/m³. In the case of deicing salts, the precise estimation of $[Cl(aq)]_0$ is difficult due to many involved parameters (i.e., frequency and quantity of salt spreading, amount of available water from rain or melted snow for salt dissolution, washout, etc.).

Parameters $[Cl(s)]_{sat}$ and K_{eq} can be determined from chloride binding isotherms. However, as a first approximation the following empirical expressions may be used (Papadakis 2000; for NaCl)

$$[Cl(s)]_{sat} = 8.8 \cdot 10^{-3} (C + k_{SF} SF + k_{FA} FA) \quad (8)$$

$$K_{eq} = 0.1 \text{ m}^3 \text{ pore volume/kg Cl} \quad (9)$$

2.2 Chloride threshold for reinforcement corrosion

It is well-established that the depassivation of embedded steel is a function not only of Cl⁻ concentration, but also of OH⁻ concentration. Diamond (1986) showed that for pH values

representative of those in concrete pore solutions (12.6-13.3) the maximum Cl^-/OH^- ratio which can be tolerated without depassivation is 0.3. This value corresponds to 0.32% total Cl^- bw of cement if the cation is Ca^{2+} and to 0.46% if the cation is Na^+ in a normal portland cement. However, much higher values of Cl^-/OH^- ratio (1.3-2) have been reported (e.g., Hussain *et al.* 1995). These disagreements are very often related to the method which applied for the pH determination, i.e., via electrode or titration, and it seems that for a safe calculation the threshold value expressed by the Cl^-/OH^- ratio should be considered as 0.6 (Goñi and Andrade 1990).

The pH-value of pore solution is determined from the presence of sodium and potassium ions rather than calcium, thus a small decrease is expected if any by the use of CH-consuming materials. In the case of fly ash this decrease is rather negligible, whereas for SF more pronounced but still higher than 12.5 for 25% replacement (Sellevold and Nilsen 1987). Taking into account the uncertainty in pH-prediction and, moreover, the rather high sensitivity of OH^- concentration on small pH changes, this approach may lead to significant different estimations for chloride threshold (e.g., pH=12.6 means $[OH^-(aq)] = 40 \text{ mol/m}^3 \text{ pore solution}$ and therefore $[Cl^-(aq)]_{cr} = 24 \text{ mol/m}^3 \text{ pore solution}$; whereas for pH = 13.6 the $[OH^-(aq)] = 400 \text{ mol/m}^3 \text{ pore solution}$ and $[Cl^-(aq)]_{cr} = 240 \text{ mol/m}^3 \text{ pore solution}$).

Another way of threshold expression is by measurement of *total* chloride ion content in concrete required for the onset of reinforcement corrosion. This approach embodies inaccuracies because only the free chlorides present in pore solution cause corrosion. However, it is very often reported that if the total chloride content is more than 0.4% by weight (bw) of cement, the steel is activated and corrosion may occur. It has been demonstrated (Sandberg 1998) that the chloride threshold for uncracked SRPC (sulphate resistant portland cement) concrete with low water-to-cement ratio, W/C (0.3-0.5) is in the range of 1-1.3% total chloride bw of binder, for SRPC concrete with 5% SF in binder is 0.8-1.0%, and with 10-20% fly ash in the binder is about 0.7%. Similar values have been reported for wider mixture proportions and materials (Xu *et al.* 2011, Yong Ann and Song 2007).

3. Application for the Øresund Link

The Øresund or Öresund Link is a combined twin-track railway and dual carriageway bridge-tunnel across the Øresund strait between Sweden and Denmark. The bridge runs nearly 8 km from the Swedish coast to the artificial island Peberholm, which lies in the middle of the strait. The remainder of the link is by tunnel from Peberholm to the Danish island Amager.

The application examined below concerns the bridge (technical information from Øresund Konsortiet 1995, Falbe-Hansen and Munch-Petersen 1998, Bræstrup and Ennemark 1998, Munch-Petersen 1998).

3.1 Estimation of parameters

Mix design parameters (bridge)

$$C = 380 \text{ kg/m}^3, \rho_C = 3130 \text{ kg/m}^3$$

$$SF = 14 \text{ kg/m}^3, \rho_{SF} = 2237 \text{ kg/m}^3 \quad (FA = 0)$$

$$W = 139 \text{ kg/m}^3, \rho_W = 1000 \text{ kg/m}^3$$

$$A = 1763 \text{ kg/m}^3, \rho_A = 2620 \text{ kg/m}^3$$

$$\varepsilon_{air} = 0.055 \text{ m}^3 \text{ of entrained or entrapped air / m}^3$$

For silica fume, a higher k_{SF} efficiency factor (about 3) as regards chloride penetration was estimated (Papadakis 2000) than the usual one (i.e., 2) reported for compressive strength, indicating that SF reduces the chloride permeability more than it improves the compressive strength. Thus, $k_{SF} = 3$ will be used in calculations.

Concrete porosity

According to Eq. (6), the total porosity is calculated as $\varepsilon = 0.116$, and the effective porosity as $\varepsilon_{eff} = 0.053$ (for 0.9 degree of hydration).

Intrinsic chloride diffusivity

According to Eq. (7), the intrinsic effective diffusivity of Cl^- in fully saturated concrete is calculated as $D_{e,Cl^-} = 5 \cdot 10^{-13} \text{ m}^2/\text{s}$. However, a lower value may be expected as Eq. (7) is valid for $W/C > 0.4$. It has also to be clarified that this diffusion coefficient calculated herein is the “intrinsic” one, completely different from the “apparent” diffusion coefficient, in which only diffusion of chlorides is taken into consideration, neglecting the significant interaction (binding) of the chlorides with the solid phase.

Chloride binding parameters

Using Eqs. (8) and (9) the saturation concentration of Cl^- in the solid phase is estimated as $[Cl(s)]_{sat} = 3.7 \text{ kg/m}^3$ concrete, and the equilibrium constant for Cl^- binding $K_{eq} = 0.1 \text{ m}^3/\text{kg}$.

Chloride threshold for reinforcement corrosion

As clearly stated above, two critical total chloride contents can be defined:

- a *pessimistic*: 0.4% by weight of cement (usually used in Denmark, by ACI etc.)
- an *optimistic*: 0.9% by weight of binder [20, SRPC concrete with 5% SF in binder]

Multiplying by the cement or binder content in concrete these thresholds are converted to:

$(\varepsilon[Cl(aq)] + [Cl(s)])_{cr,min} = 1.5 \text{ kg total chlorides/m}^3 \text{ concrete}$, and

$(\varepsilon[Cl(aq)] + [Cl(s)])_{cr,max} = 3.5 \text{ kg total chlorides/m}^3 \text{ concrete}$,

3.2 Exposure conditions

As stated in technical note (Ø resund Konsortiet 1995), the concentration of Cl^- in sea water in Ø resund is between the concentration in the North Sea (16 kg/m^3 solution) and the concentration in the Baltic Sea (4 kg/m^3 solution). The value of 16 kg/m^3 was selected in those calculations. In the present approach both cases will be presented.

In the case of deicing salts high chloride concentrations of 100 kg/m^3 solution can be considered. Taking into account that the salt spreading takes place few months per year and moreover due to wash by rain, the chloride surface concentration decreases, as a first approximation an equivalent exposure to a 16 kg/m^3 (as for North Sea) continuously all year around can be considered. However, a more realistic approach based on statistical data should be sought.

Another important conclusion from the present model parametric analyses (Papadakis *et al.* 1996) should be referred herein. Let us suppose that the exposure of the concrete surface to chlorides is not continuous but periodic, dividing the total period of time (say t_{max}), into a number of intervals of length T , where during one part rT of which the surface is considered to be exposed to

chloride ingress, while during the rest, $(1-r)T$, it is not. Pore saturation conditions, though, were not considered to be affected by the change in exposure conditions. Results (Papadakis *et al.* 1996) showed that the free chloride concentration at any time and distance is independent of total duration T of the exposure-nonexposure cycle and it can be calculated multiplying that of the continuous exposure by the exposure degree r , i.e.

$$[Cl(aq)]_r = r [Cl(aq)]_{r=1} \quad (10)$$

3.3 Model predictions and discussion

The Eq. (1) was solved numerically using a finite difference method. The EUCON[®] software, developed by the present author (Papadakis *et al.* 2007), was used. Fig. 1 shows the time evolution of the free-Cl⁻ concentration, $\varepsilon[Cl(aq)]$, and the total-Cl⁻ one, $\varepsilon[Cl(aq)] + [Cl(s)]$, as a function of the distance from the outer concrete surface, for a saturated slab continuously exposed to Baltic sea water. Cl⁻ profiles for 10, 25, 50, 75 and 100 (t_{max}) years are shown. The exposure to North Sea water (or equivalently an exposure to deicing salts) is presented in Fig. 2.

In the case of *Baltic Sea exposure* (Fig. 1), for this specific concrete and deterioration mechanism, there is no problem for a period of 100 years and for all concrete covers, even if the minimum critical value is considered.

In the case of *North Sea exposure* (Fig. 2), different results are obtained depending on critical value considered. Denoting by L (years) the time required for chloride-induced bar depassivation (practically, a safe estimation of concrete service lifetime) and c (mm) the reinforcing bar distance from outer concrete surface (cover), the relation of L - c can easily be obtained from Fig. 2 and it is presented in Fig. 3. It is observed that for 100 years service life a minimum concrete cover of:

- 45 mm is required, if the maximum critical value is considered (3.5 kg/m^3) or
- 110 mm is required, if the minimum critical value is considered (1.5 kg/m^3).

It is observed that the service lifetime is proportional to the square of concrete cover as given in Fig. 4. From regression analysis the following equations are obtained:

$$L = 8.214 \cdot 10^{-3} c^2 \quad (\text{minimum critical value}) \quad (11)$$

$$L = 53.21 \cdot 10^{-3} c^2 \quad (\text{maximum critical value}) \quad (12)$$

The coefficient of determination, R^2 , is very close to unity, 0.99 and 0.98 for Eqs. (11) and (12), respectively. According to initial specifications (Ø resund Konsortiet 1995, Falbe-Hansen and Munch-Petersen 1998, Bræstrup and Ennemark 1998, Munch-Petersen 1998) a minimum cover of 75 mm is required for 100 years service lifetime. This dependence is expressed by

$$L = 17.778 \cdot 10^{-3} c^2 \quad (\text{proposed}) \quad (13)$$

• It is pleasantly observed, that the proposed, and already used, relationship of service lifetime-cover is almost identical with a mean line between the lines derived from the minimum and maximum critical values considered for corrosion initiation. Thus, an *excellent agreement with the project specifications* is observed despite the different ways used to approach the problem. It means that the models used in the paper fit the prescriptive specifications. Project specifications came from field data in North Sea exposure, and thus this agreement offers a significant validation in the models.

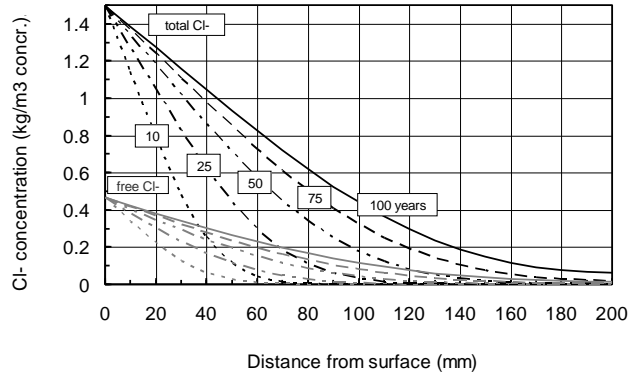


Fig. 1 Chloride penetration profiles for exposure to Baltic Sea water (4 kg/m^3 solution)

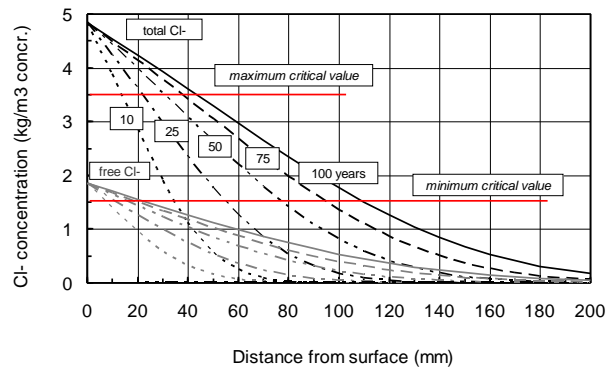


Fig. 2 Chloride penetration profiles for exposure to North Sea water or equivalently to deicing salts (16 kg/m^3 solution)

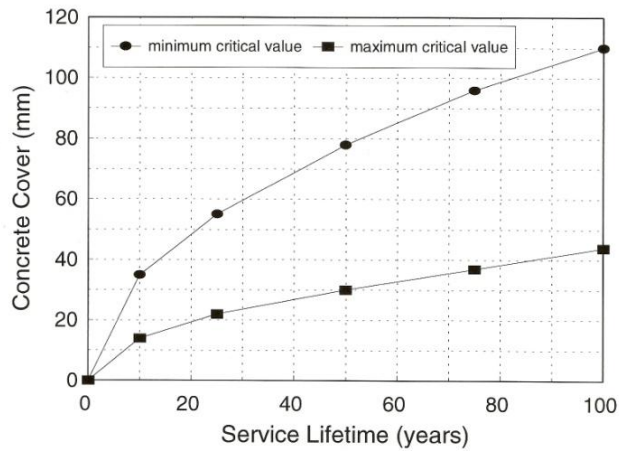


Fig. 3 Concrete cover- service lifetime relationship (North Sea/ deicing salts exposure)

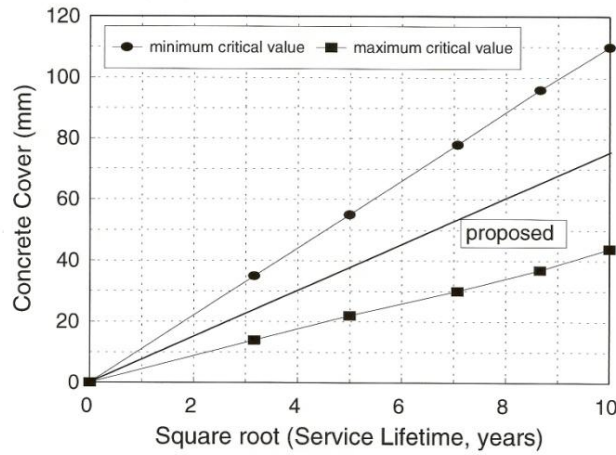


Fig. 4 Concrete cover- square root of service lifetime relationship (North Sea/ deicing salts exposure)

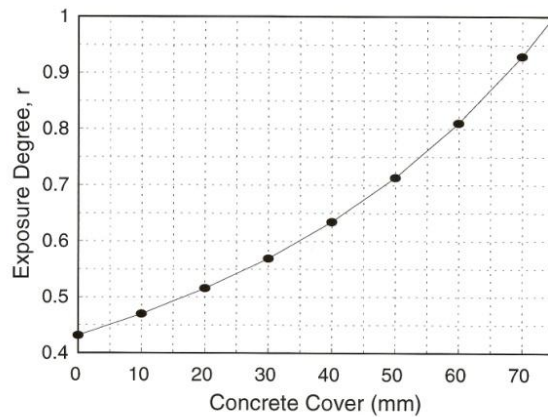


Fig. 5 Maximum exposure degree as a function of concrete cover which ensures service lifetime 100 years (North Sea/ deicing salts exposure)

• Taking into account that the above Eq. (13) is a first approximation to this very complicated problem, it can rather precisely be used for comparison reasons. Another important observation is that because the overall process of chloride ingress in concrete is a diffusion-controlled process, there is a proportionality of depth (concrete cover) with the square root of time (service lifetime), see Fig. 4.

3.4 Scenario of concrete cover failure- Application

As stated above no significant problem should be expected if concrete cover is greater than 75 mm, even in the case when no coating is used. When cover is less than 75 mm a decrease in lifetime is expected according to Eq. (13), in the absence of protective coating.

The concrete surface is going to be covered by an *asphalt membrane* in order to protect the

structure from any concrete cover failure. However, this coating can be considered waterproof only for some years, say X . Then, the chlorides through coating holes can easily attack the concrete for some years, say Y . Then, a repair takes place which will protect the concrete for X years, etc.

Let us suppose that the number of repairs is n within the designed service lifetime (a small number of 2-5 should be expected). Thus

$$L = n(X + Y) \quad (14)$$

$$r = Y / (X + Y) \quad (15)$$

where, r is the exposure degree (see above- exposure conditions' section). As mentioned in Section 3.2, the free chloride concentration at any time and distance is independent of the number of repairs, n , and it can be calculated multiplying that of the continuous exposure by the exposure degree r , see Eq. (10). Thus, the free chloride concentration at the designed service lifetime L at any distance from concrete surface is given by

$$\varepsilon[Cl^{-}(aq)]_{r,L} = \varepsilon r [Cl^{-}(aq)]_{r=1,L} \quad (16)$$

In order to ensure reinforcing bar protection at a given cover c , the following requirement, Eq. (17), should be fulfilled for the total chloride concentration (which equals to $\varepsilon [Cl^{-}(aq)]_{r,L} + [Cl^{-}(s)]_{r,L}$; where $[Cl^{-}(s)]_{r,L}$ is calculated from Eq. (2). The mean critical value of 2.5 kg total chlorides/ m³ concrete is used as demonstrated above, Section 3.1.

$$\varepsilon r [Cl^{-}(aq)]_{r=1,L} + \frac{K_{eq} r [Cl^{-}(aq)]_{r=1,L}}{1 + K_{eq} r [Cl^{-}(aq)]_{r=1,L}} [Cl^{-}(s)]_{sat} \leq 2.5 \quad (17)$$

From the above Eq. (17), it is possible to estimate the maximum exposure degree r by having already calculated the free chloride concentration at the case of continuous exposure ($r = 1$) at the designed service lifetime of $L = 100$ years, and knowing the ε , K_{eq} , and $[Cl^{-}(aq)]_{sat}$ values. Following this procedure, the dependence of concrete cover c on maximum exposure degree r is estimated and presented in Fig. 5. It is observed that for concrete cover $c = 75$ mm the maximum exposure degree r equals 1 which means that even in this case of absence of protective coating a full protection is ensured for 100 years. On the other hand, even in the case of zero concrete cover, a maximum exposure degree r equals to 0.44 is estimated which means that if the coating protection is at least secure for 56% of the whole period then no corrosion may occur. In the following application intermediate cases are presented.

Application

As the problem is independent of the number of repairs, any n can be selected, say $n = 5$, thus in a designed service lifetime of 100 years, every 20 years a repair of asphalt membrane will take place. Let us suppose that a mean concrete cover in a part of the structure is 30 mm. According to Fig. 5, a maximum exposure degree of $r = 0.57$ is calculated. This means that the asphalt membrane should be completely waterproof for at least $(1-0.57) \cdot 20 = 8.6$ years within the 20-year period. If $n = 2$ is selected, then the asphalt should be waterproof for 21.5 years in a 50-year period.

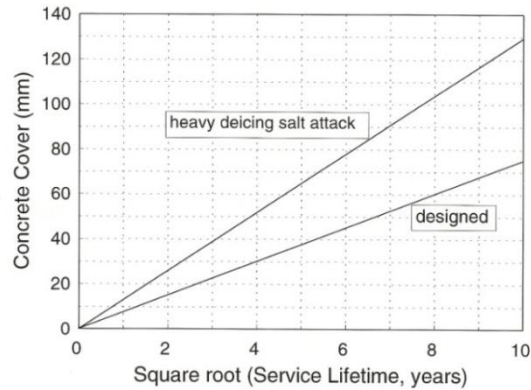


Fig. 6 Comparison between proposed prediction of service life (exposure to North Sea water) and an extreme case of heavy deicing salts attack

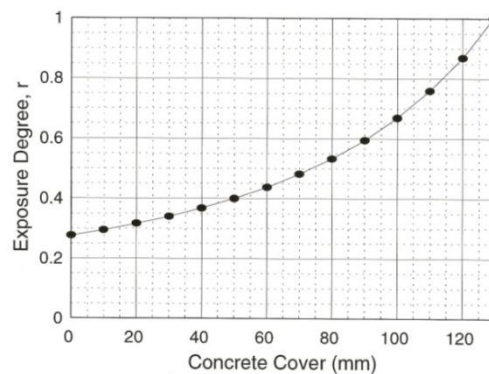


Fig. 7 Maximum exposure degree as a function of concrete cover which ensures service lifetime 100 years (extreme case of heavy deicing salts attack)

3.5 Extreme deicing salts attack - Application

Let us consider a chloride attack case due to an *extremely high* Cl^- -concentration and duration 4 of deicing salting. A chloride concentration of 100 kg/m^3 solution is considered. It is assumed that the salt spreading takes place continuously 3 months per year, whereas the remaining 9 months the surface is exposed to zero salt concentration. Solving the Eqs. (1)-(4) for the case of continuous exposure to 100 kg/m^3 surface Cl^- -concentration, the free and total chloride concentrations are calculated as a function of distance and time. The free-chloride concentration is multiplied by the exposure degree $r = 3/12$, in order to convert it in the case of interrupted exposure, following exactly the same findings of Papadakis *et al.* (1996), expressed by Eq. (10). The corresponding solid-chloride concentration is calculated from the Eq. (2) and from their sum, the total chloride concentration. Taking as a critical value for corrosion the mean value of 2.5 kg/m^3 , the dependence of predicted service life on concrete cover is illustrated in Fig. 6. A very pessimist prediction is observed for this extreme case, as compared to the proposed one.

Following exactly the same procedure as above (3.4), the maximum exposure degree ratio, r , is

calculated for this extreme case and presented in Fig. 7. Thus, for the case of designed service lifetime of 100 years, repair number of 5 (every 20 years a repair of asphalt membrane will take place), and mean concrete cover of 30 mm, a maximum exposure degree of $r = 0.34$ is calculated. This means that the asphalt membrane should be completely waterproof for at least $(1-0.34) \cdot 20 = 13.2$ years within the 20-year period.

4. Conclusions

In the present work, a simulation tool, based on proven predictive models utilizing principles of chemical and material engineering, for the estimation of concrete service life is applied on an existing reinforced concrete bridge (Øresund Link) located in a severe environment. By taking under consideration the concrete compositional parameters, the environmental conditions and various structural properties of the bridge investigated, an accurate prediction of its service life is taking place.

In the case of *Baltic Sea exposure*, for this specific concrete and deterioration mechanism, it was found that *there is no problem* for a period of 100 years and for all concrete covers, even if the minimum critical value is considered.

In the case of *North Sea exposure*, different results are obtained depending on critical value considered. It is observed that for 100 years service life a minimum concrete cover of:

- 45 mm is required, if the maximum critical value is considered (3.5 kg/m^3) or
- 110 mm is required, if the minimum critical value is considered (1.5 kg/m^3).

It was observed that the service lifetime is proportional to the square of concrete cover. It was also observed, that the proposed, and already used, relationship of service lifetime- cover is almost identical with a mean line between the lines derived from the minimum and maximum critical values considered for corrosion initiation. Thus, an *excellent agreement with the project specifications* is observed despite the different ways used to approach the problem. The application of the already presented models to a real bridge and the observation of an excellent agreement with the project specifications is a new finding of the present work. Project specifications came from field data in North Sea exposure, and thus this agreement offers a significant validation in the models.

Furthermore, different scenarios of concrete cover failure, in the case when a coating is utilized and extreme deicing salts attack are also investigated. Overall various protective measures are suggested for secure protection against chloride attack. This further investigation on protection measures and failure, which are approached for first time by the suggested models of the author, is another new finding of the present work.

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