Deterioration in quay-wall structures and effect of hydrophobic treatment

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Abstract

Chloride ingress is one of the main causes of corrosion of the reinforcement in concrete structures, certainly for structures that are highly exposed to chlorides, such as off-shore or on-shore constructions in marine environment. This paper deals with the quay-wall of the new container terminal at Zeebrugge Harbor, Belgium, which is partly protected against chloride ingress by means of a water repellent agent immediately after construction in 1993. To judge the in situ effectiveness of the hydrophobic agent as a water repellent treatment, three subsequent in situ surveys have been conducted in 1996, 1998 and recently in 2005. Based on the cores drilled, the chloride profiles are determined as a function of time, both in a non-treated and treated location. The results are presented and discussed. Because of the long-term data sequence, the long-term effectiveness of the treatment can be assessed in an objective way.

1. Introduction

Chloride ingress, due to capillary suction or diffusion, in combination with carbonation are two of the main causes of corrosion of the reinforcement in reinforced concrete structures. This paper deals with the quay-wall of the new container terminal at Zeebrugge Harbor, Belgium. This type of structure is subjected to chloride ingress, not only in the submerged area, but also in the tidal zone, the splash zone and in the atmospheric zone on top of the quay-wall. Those structures can be protected by means of a protective coating or by a water repellent agent.

In order to increase the durability of the concrete, it was decided to apply an hydrophobic treatment on the new quay-wall of the container terminal at Zeebrugge. The construction of the container terminal at Zeebrugge was ordered by the Ministry of the Flemish Community, Sea-Harbor Division and finished in 1993. Figure 1 gives a general view of the quay-wall, constructed on top of cylindrical sunk down reinforced cells (caissons).

The effectiveness of water repellent agents has been studied in the Reyntjens Laboratory of KULeuven (Van Tongelen et al., 1994; Van Gemert, 1992). Based on experimental results, it was decided in 1993 to apply Protectosil® BHN (former Dynasylan® BH N: alkyltriethoxysilane). For practical reasons and an ease application, an hydrophobic treatment with highly concentrated solvent-free compounds based on alkyltriethoxysilane, was used in order to prevent the wall for damage caused by chloride penetration, pitting corrosion and alkali-aggregate-reaction. At the construction site, the following application scheme has been adopted:

- a first application of the water-repellent was done immediately after demolding. This prevented the surface drying of the concrete as well as the eventual penetration of salt water;
- a second application was executed 7 days after demolding, in order to obtain a deep penetration in the concrete.
The waterproofing has been applied by means of airless spraying at low pressures, using a plunger pump. The consumption was measured to be 0.35 litres per m² (300 g/m²), which coincided very well with the preliminary tests in the laboratory on concrete cubes, prepared on the construction site.

In this specific case however, a part of the quay-wall has not been treated with the water repellent agent, which offers the unique opportunity of spatial comparison of the deterioration of the concrete cover, both related to chloride ingress and carbonation depth.

Figure 1: Plan view and cross section of the quay-wall at Zeebrugge Harbor

During three subsequent test campaigns - 1996, 1998 and 2005 after 3, 5 and 12 years of in site service - cores are drilled at different locations and in treated and non-treated zones. From these cores, chloride profiles and carbonation depths are determined (Schueremans and Van Gemert, 1996, 1998, 2005). This allows comparison of the effectiveness of the hydrophobic agent at a certain moment in time and at a specific location (Schueremans and Van Gemert, 1999a,b; Schueremans et al., 2005; Schueremans et al., 2006). For the three subsequent test campaigns, cores have been drilled at neighboring positions. As a result, the chloride ingress and carbonation can be determined as a function of time and place, both in the non-treated and treated location.

The obtained data are used to feed the deterioration models related to the chloride ingress into the structure and to the increase of carbonation depth. The environmental loading originating from the tidal effect of the North Sea will differ for the parts of the quay-wall in tidal zone, splash zone or atmospheric zone, on top of the quay-wall, were the test samples have been taken. As a result, the spatial variation of the failure probability for the durability limit state function related to the preset service life is estimated. The latter are compared with general accepted target failure probabilities.

2. Test campaign – in site and laboratory tests

During each of the three subsequent in site surveys, cores have been drilled at following locations:
During these in situ campaigns, concrete cores (diameter = 50mm) are drilled at neighbouring positions. In general, 3 types of tests are performed on the cores drilled:

- on all cores the penetration depth of the hydrophobic agent is determined based on a visual interpretation, by wetting a fresh cracked surface, or based on pyrolysis gaschromatography;
- on all cores the carbonation depth is measured before the cores are sawn into slices, on the site directly after drilling as well as in the laboratory the day after drilling for comparison;
- after cutting into slices, water soluble and acid soluble chloride contents are measured at several depths to be able to estimate a chloride penetration profile at different locations.

### 2.1. Carbonation depth

The carbonation depths are determined by treating the fresh sawn surfaces with a phenolphthaleine solution (dissolved in 1% ethyl alcohol). The values are listed in table 1. The carbonation depth remains approximately zero in the non-treated location. In the treated location the carbonation depth varies from 4-6 mm in tidal zone, towards 8-12 mm above tidal zone and up to 12-16 mm on top of the quay-wall.

Comparing the different locations, it is clear that the diffusion of CO₂ into the concrete is improved causing carbonation since the entrance of water is prevented by means of the hydrophobic agent.

#### Table 1: Carbonation depth in non-treated and treated locations

<table>
<thead>
<tr>
<th>Location</th>
<th>2005</th>
<th>1998</th>
<th>1996</th>
</tr>
</thead>
<tbody>
<tr>
<td>Splash zone in tidal zone</td>
<td>0</td>
<td>0-0.5</td>
<td>0</td>
</tr>
<tr>
<td>A: non-treated</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Splash zone in tidal zone</td>
<td>0</td>
<td>0</td>
<td>1</td>
</tr>
<tr>
<td>B and C: treated with Protectosil® BHN</td>
<td>8-12</td>
<td>6-12</td>
<td>4</td>
</tr>
<tr>
<td>Splash zone in tidal zone</td>
<td>4-6</td>
<td>5-10</td>
<td>5</td>
</tr>
<tr>
<td>Atmospheric zone</td>
<td>12-16</td>
<td>5-10</td>
<td>5</td>
</tr>
</tbody>
</table>

### 2.2. Chloride concentration profiles

Two different values of chloride concentration have been determined: the water soluble chloride content and the acid soluble chloride content. To determine the chloride penetration profiles, the cores were sawn in slices and both types of chloride content have been determined at several depths on slices of average thickness of 7 mm. The chloride contents were determined by means of wet chemical analysis, according to the Belgian Standard NBN B15-250 (1990). The chloride content, obtained by wet chemical analysis, equals the amount of free chlorides and a great deal of the chlorides, bound under the form of Salt of Friedel which dissolves in the water during extraction. Since the water soluble chlorides represent the chloride ions endangering the steel to corrode, these are reported here solely. The experimental results (%Cl⁻/cem) are presented in figures 2 up to 6, representing the chloride concentration at the above mentioned 5 locations as a function of time and depth. Each of the values, plotted in the figures, is the average of 3 comparative chemical analyses. With respect to the presented chloride penetration profiles, some remarks have to be made regarding the accuracy of the experimental results. The data are subject to
scatter due to the heterogeneous composition of concrete - aggregates and cement paste as well as the relatively small size of the samples analysed (cores with diameter of 50 mm and slices of concrete with an average thickness of 7 mm). A variation in results might originate from the number and size of aggregates within the tested sample.

**Figure 2:** water soluble chloride content - location A - non-treated - splash zone (1996-1998-2005);

**Figure 3:** water soluble chloride content - location A - non-treated - in tidal zone (1996-1998-2005);

**Figure 4:** water soluble chloride content - Location B - treated with Protectosil® BHN - splash zone (1996-1998-2005);

**Figure 5:** water soluble chloride content - Location B - treated with Protectosil® BHN - in tidal zone (1996-1998-2005);

**Figure 6:** water soluble chloride content - Location C - treated with Protectosil® BHN - atmospheric zone (1996-1998-2005);

**Figure 7:** Critical chloride content as a function of Relative Humidity (Gehlen and Schiessl, 1999; Song et al., 2005)

### 3. Probabilistic modelling

For the probabilistic modeling, the focus is on the chloride ingress and carbonation depth. For both degradation phenomena, only the initiation periods are taken into account. Their respective limit state functions reflect the carbonation front reaching the concrete cover depth and the chloride concentration reaching the critical value at the location of the rebar. The first one leads to depassivation of the concrete cover at the rebar position, the latter fits the end of the initiation period, at which corrosion of the rebar might start. The respective limit state functions read:

\[
g_1 = x_c - d(t) \\
g_2 = C_{cr} - C(x_c, t)
\]

In which \(C_{cr}\) is the critical chloride content which strongly depends upon the humidity (Song et al., 2005). Overall, in case of frequently changing humidity (splash zone, tidal area), a normal probability density function with an average value equal to \(\mu=0.50\%\text{Cl}/\text{cem}\) and standard deviation \(\sigma=0.10\%\text{Cl}/\text{cem}\) is preset for concrete with a water on cement-ratio close to \(w/c=0.50\), Figure 7. In the presence of algae on the concrete cover, the surface remains wet and the drying period is almost non-existent. In case of water saturated surface, with a constant humidity that approaches 100%, the critical chloride concentration will be significantly higher. An average value of \(\mu(C_{cr})=0.70\%\) will be used (Gehlen and Schiessl, 1999). For the locations treated with a water repellent agent, the humidity is kept out strongly, which is evidenced from the increased carbonation front, see further. Also in this case an average value of \(\mu(C_{cr})=0.70\%\) will be used. The standard deviation is kept constant, inducing a reduction in the coefficient of variation as can be seen from Figure 7. \(X_c\) is the concrete rebar cover. To account for the geometrical uncertainty, a lognormal PDF is assumed, with a 15% coefficient of variation. \(d\) is the carbonation depth.
Both durability and serviceability limit states are treated separately since information on the impact of combined loading is lacking data. A target reliability equal to $\beta_T=1.5$ is preset (Schiessl and Gehlen, 2005). This value is still subject of discussion. At this moment, there is no reliability differentiation depending on possible hazard scenarios, economic constraints and expected consequences of failure.

3.1. Modeling of carbonation front

In general the carbonation depth is modeled using suitable modifications of Fick’s 1$^\text{st}$ law of diffusion, following:

$$d(t) = \sqrt{\frac{2D_c C_s}{a}} \sqrt{t} = k \sqrt{t}$$

(3)

<table>
<thead>
<tr>
<th>Location Description</th>
<th>Based on 1996, 1998 and 2005 survey data [m/s(^{1/2})]</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$\mu(k) \times 10^{-5}$ [m$^2$/s]</td>
</tr>
<tr>
<td>A (non-treated): splash zone</td>
<td>5</td>
</tr>
<tr>
<td>A (non-treated): tidal zone</td>
<td>0</td>
</tr>
<tr>
<td>B (treated): splash zone</td>
<td>58</td>
</tr>
<tr>
<td>B (treated): tidal zone</td>
<td>40</td>
</tr>
<tr>
<td>C (treated-on top of quay-wall): atmospheric zone</td>
<td>62</td>
</tr>
</tbody>
</table>

In which $D_c$ is the carbon dioxide diffusion coefficient [m$^2$/s], $C_s$: the carbon dioxide concentration at the surface [kg CO$_2$/m$^3$], $a$ the binding capacity [kg CO$_2$/m$^3$] and $t$ the age of the concrete [s]. Since experimental data are available, all parameters are stored in a single carbonation rate ($k$) [m/s$^{1/2}$]. The latter inherently accounts for all the uncertainties related to the surface loading, the binding capacity and the concrete diffusivity for carbon dioxide, which individually are extremely complex to measure including effects of ageing. The carbonation rate is assumed to be lognormal distributed. Their parameters, based on the experimental data, are listed in Table 2.

3.2. Modeling of chloride ingress – transport/degradation model

It is virtually impossible to introduce a mathematical model taking into account all the variables involved in the corrosion process. In general two different types of transport models are used (BE95-1347/R4-5, 1998):

- empirical models based on modified versions of Fick’s 2$^\text{nd}$ law. Fick’s second law describes the transport of chlorides in the concrete due to diffusion. In reality, the diffusion process is only valid in saturated conditions. When the pores are empty, capillary forces drag the outside solution into the concrete, bringing the salts along. Other assumptions are made that are not completely valid for concrete. In the derivation of Fick's law it is assumed that the porous material is homogeneous, that the medium is non-reactive and non-adsorptive, which are not valid for concrete. Despite of the differences between the assumptions on which Fick's law is based and reality for the application intended, it is still accepted as the common model to describe chloride transport into concrete;

- quasi-scientific models that rely on a scientific approach in modelling the chloride penetration process addressing simultaneously diffusion and (ir-)reversible chemical reactions (Crank, 1975). At present, few models are available, all relying on simplifying assumptions and involving several parameters for which experimental data is missing.
Of course, a similar analysis could be performed on more accurate models as they become available. The transport model and its limitations also count for a classical deterministic analysis and certainly is not a limitation of the probabilistic analysis.

The diffusion law, Fick's 2nd law for one-dimensional chloride diffusion into concrete, takes the form:

\[
\frac{\partial C}{\partial t} = \frac{\partial}{\partial x} \left(D \frac{\partial C}{\partial x}\right) \tag{4}
\]

in which concrete is assumed to be a homogeneous, isotropic material. When assumed that no reaction occurs between concrete and chlorides, an explicit solution of this differential equation can be obtained, using the following boundary conditions:

- \(C(x=0,t) = C_0; 0<x<\infty\) (the initial chloride concentration in the concrete mix) and
- \(C(x=0,t) = C_{Sa}; 0<t<\infty\) (the achieved surface chloride concentration loading at the surface of the concrete due to the marine environment).

\[
C_i(x,t) = C_0 + (C_{Sa} - C_0) \text{erfc} \left( \frac{x}{2\sqrt{D_a t}} \right) \tag{5}
\]

in which \(C_i(x,t)\) is the amount of chlorides at time \(t\) at a distance \(x\) from the concrete surface, \(D_a\) the achieved chloride diffusion coefficient and \(\text{erfc}()\) is the error function. In literature the achieved chloride diffusion coefficient \((D_a)\) is often assumed time dependent \((D_a(t))\), following (BE95-1347/R4-5, 1998):

\[
D_a(t) = D_a(t_0) \left( \frac{t_0}{t} \right)^n k_t k_c k_e \tag{6}
\]

In which \(D_a(t_0)\) is the initial diffusion coefficient after curing time \((t_0)\), \(k_t\) is a testing factor, \(k_c\) is a curing factor and \(k_e\) is an environmental factor. All correction factors \(k\) are set equal to 1 since direct testing on the structure is performed. Additionally, the time dependency is not accounted for, since, at this moment in time, there is no experimental evidence from the determined chloride profiles that ageing has affected the diffusion coefficient, both for the treated as non-treated area. Further more, since the transport model is a simplification of reality, the diffusion coefficient should be interpreted as a transport or migration parameter. In that it also accounts for the chloride barrier caused by the hydrophobic agent.

For the environmental action, different zones need to be distinguished:

- Submerged zone (no data available from in site testing);
- Tidal zone;
- Splash zone;
- Atmospheric zone (i.e.: on top of the quay-wall).

In general the spatial distribution of the surface chloride content \((C_s)\), can be written following (BE95-1347/R3, 1999)

\[
C_s = C_{s,0} f(h,z) \tag{7}
\]
In which \( C_{S,0} \) is the nominal surface chloride concentration and \( f(h,z) \) accounts for the effect of the height \( (h) \) and distance \( (z) \) from the sea shore. For the vertical quay-wall, the environmental loading in tidal and splash zone can be rewritten into (BE95-1347/R3, 1999):

\[
C_S = C_{sea} f(h)
\]

In which \( C_{sea} \) is the concentration of chlorides in the sea (3.5m\%Cl/H\_2O) and \( f(h) \) is the height above the tidal zone. Data on the spatial distribution is scarce, but it has been verified that in general a decrease is only seen from about 15 m above the water level (Sørensen and Maahn, 1982).

Further discrepancy with the proposed formula appears from the measured chloride concentrations at the surface layers. As can be derived from Figures 3 till 8, the water soluble chloride content as measured at depths of 0-9 mm and 10-20 mm exceed in many cases 3,5 percent by weight of water. Comparable results can be found elsewhere (Gonzales et al., 1996a,b). Since the inversion of Fick’s second law is only soluble when \( C_0 < C_i(x,t) < C_S \), a least square optimization proved that a value of \( C_S = 7 \text{ m}\% \text{Cl/H}_2\text{O} \) results in chloride profiles that are nearest to the measured profiles, with a close overlap at largest depths which is of main interest. The higher value can be explained by the presence of algae at the concrete surface in the tidal zone, of which the chloride concentration can reach up to 9 percent by weight of water, and by salt crystallization (Costa & Appleton, 1999) and are in line with the higher values reported elsewhere (BE95-1347/R9, 2000).

For the atmospheric zone, on top of the quay-wall, the environmental chloride loading can be written as:

\[
C_S = C_{air} f(h,z)g(w)
\]

In which \( C_{air} \) is the maximum equivalent concentration of chlorides in the air, and \( g(w) \) accounts for the effect of orientation of the concrete surface towards salt mist and driving rain. More explicit information on both the distance function \( f \) and the orientation function \( g \) is lacking in literature. Since the investigated surface is a horizontal plane, the latter is left out. Additional complexity arises in determining \( C_{air} \). From the measured chloride profiles indeed, high surface concentrations are required to retrieve comparable values for the diffusion coefficient at location B and C, both constructed and treated under similar circumstances. This can be explained by the fact that the data are collected from a horizontal surface, at which during ongoing cycles of wetting and drying, the surface chloride concentration increased through storage capacity in the concrete pore system. This is further exemplified by the apparent increase of diffusion coefficient in this area adding the data from the 2005 survey, back-calculated from an assumed constant surface loading, set equal to the values used in the splash and tidal zone, Table 3. This is significantly higher then the maximum surface concentration of chlorides in the atmospheric zone reported in literature (BE95-1347/R9, 2000), that equals: 2.5-3.0\%\text{Cl/H}_2\text{O}.

The initial chloride concentration in the concrete mix (\( C_0 \)) is taken from comparable concrete mixes from neighbouring marine constructions at their erection time, as there were no test results available of the original chloride content from the quay-wall itself: \( C_0 = 0.02\text{\% Cl/} \text{cem} \).

Table 3 gives the mean value \( \mu(D) \) and the standard deviation \( \sigma(D) \) for the groups of cores taken at different locations during the 3 subsequent surveys in 1996, 1998 and 2005. The first columns reflect the obtained values of the diffusion coefficient and predicted service life after 5 years in site exposure; the subsequent columns present the updated values after 12 years of in site exposure, also accounting for the survey performed in 2005. The diffusion coefficient is assumed to be lognormal distributed (Prezzi et al., 1996). The coefficient of variation on the diffusion coefficient is significantly larger then the overall value of 28.5% reported in literature (BE95-1347/R9, 2000).
Table 3: Diffusion coefficient $D_a$ – parameters of lognormal distribution

<table>
<thead>
<tr>
<th>Location</th>
<th>Based on 1996 and 1998 survey data $[m^2/s \times 10^{-12}]$</th>
<th>Based on 1996, 1998 and 2005 survey data $[m^2/s \times 10^{-12}]$</th>
</tr>
</thead>
<tbody>
<tr>
<td>A (non-treated)</td>
<td>$\mu(D_a)$: 9.64, $\sigma(D_a)$: 7.0</td>
<td>$\mu(D_a)$: 9.58, $\sigma(D_a)$: 10.55</td>
</tr>
<tr>
<td>B (treated)</td>
<td>$\mu(D_a)$: 1.18, $\sigma(D_a)$: 1.0</td>
<td>$\mu(D_a)$: 1.61, $\sigma(D_a)$: 2.49</td>
</tr>
<tr>
<td>C (treated-on top of quay-wall)</td>
<td>$\mu(D_a)$: 1.32, $\sigma(D_a)$: 1.7</td>
<td>$\mu(D_a)$: 2.13, $\sigma(D_a)$: 3.46</td>
</tr>
</tbody>
</table>

3.3. Reliability analysis

Table 4 summarizes the random variables used in the analysis, related to both limit state functions, eq. 1 and 2. Based on this information the service life (initiation) period is estimated at which the threshold reliability $\beta_T=1.5$ or corresponding failure probability $p_{f,T}=0.067$ is reached. Additionally, the risk for corrosion initiation is calculated for the actual service life, which is 12 years. The results are outlined in Table 5.

Table 4: (Random) variables and their parameters

<table>
<thead>
<tr>
<th>Random variable</th>
<th>PDF</th>
<th>Mean $\mu$</th>
<th>Standard Deviation $\sigma$</th>
<th>Cov [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td>$x_c$</td>
<td>Concrete cover [mm]</td>
<td>Lognormal</td>
<td>Nominal:120</td>
<td>18</td>
</tr>
<tr>
<td>$C_{cr}$</td>
<td>Critical chloride content [%Cl-/cem]</td>
<td>Normal</td>
<td>0.70</td>
<td>0.10</td>
</tr>
<tr>
<td>$k$</td>
<td>Carbonation rate $[m/s^{1/2}]$</td>
<td>See Table 2</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$D_a$</td>
<td>Diffusion coefficient $[m^2/s \times 10^{-12}]$</td>
<td>See Table 3</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 5: Service Life (SL) prediction for target reliability index - $\beta_T=1.5 - p_{f,T}=0.067$

<table>
<thead>
<tr>
<th>Service Life Prediction value [year] versus Location</th>
<th>LSF 1: depassivation of concrete cover at rebar due to carbonation $g_1 = x_c - d(t)$</th>
<th>LSF 2: initiation of chloride induced corrosion $g_2 = C_{cr} - C(x_c, t)$</th>
<th>SL at $\beta_T=1.5$</th>
<th>SL at $\beta_T=1.5$</th>
<th>$\beta (p_f)$ at SL=12 year</th>
</tr>
</thead>
<tbody>
<tr>
<td>A: non-treated: Splash zone Tidal zone</td>
<td>371 / [beyond accuracy of data]</td>
<td>3.5</td>
<td>0.23 (0.4)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>B: treated: Splash zone Tidal zone</td>
<td>98</td>
<td>19</td>
<td>1.9 (0.03)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>C: treated: atmospheric zone</td>
<td>97</td>
<td>17.5</td>
<td>1.7 (0.04)</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

From Table 5 it is clear that:

- No or limited carbonation risk is observed in the tidal zone. The risk for depassivation of the rebars is increasing when going to increased aerated areas, splash zone and atmospheric zone;
- The use of a water repellent agent increases the carbonation rate and influences the risk on depassivation. In all cases, the preset service life remains sufficient;
- With respect to the service life for the preset target reliability ($\beta_T=1.5$), the difference in between the non-treated and treated area is significant, demonstrating the effectiveness of the hydrophobic agent decreasing the risk for chloride induced corrosion;
- The service life prediction for the atmospheric zone is comparable with the tidal and splash zone, since the surface loading as well as the critical chloride content used in the analysis are similar. The effect of the water repellent agent on the humidity in the concrete and its relation with the critical chloride content however are not evidenced with experimental data nor can be retraced from literature;
• The last column of Table 5 presents the spatial distribution for the risk on corrosion initiation. In practice, even in the non-treated location with a probability of 40% on corrosion initiation, no corrosion is observed in the investigated areas.

Discussion and conclusions

In none of the cases listed in Table 5, the preset target reliability is obtained for a service life of 50 or 100 years. This is generally the case for structures built with ordinary Portland cement (BE95-1347/R9,2000) and is in line with comparative results of benchmark reliability calculations (BE95-1347/R12-13, 2000). In countries were the use of blast furnace slag cement or Portland cement with fly ash are common practice, such as in the Netherlands, the target reliability level (β_T=1.5) is reached generally because of the more favourable value of the diffusion coefficient. However, in practice, it can be verified that up till now in none of the investigated locations, the main rebars are corroding. Thus, the obtained results are to be interpreted as a percentage of risk on corrosion. Although only the initiation period is accounted for, the target reliability values for the serviceability or durability limit states are used. This is conservative, resulting in limited service life prediction values. This can be put to question. Less severe target values could be put forward. Further, there is an overall lack of data that needs to be solved, such as on reliable values for the critical corrosion content, surface loading and the actual parameters for the spatial distribution of these values. Finally, it is stressed that for the moment only the individual effects of carbonation or chloride ingress are treated. The combined effect of carbonation on the diffusion coefficient or vice versa is not accounted for. This requires more fundamental research on the transport models, for which simplified empirical models are used at this stage.

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References


