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Publication date
2016

Document Version
Accepted author manuscript

Published in
Life-Cycle of Engineering Systems: Emphasis on Sustainable Civil Infrastructure - 5th International Symposium on Life-Cycle Engineering, IALCCE 2016

Citation (APA)
van Breugel, K., Polder, R. B., & de Rooij, M. R. (2016). Long-term performance of marine structures in the Netherlands - validation of predictive models for chloride ingress. In *Life-Cycle of Engineering Systems: Emphasis on Sustainable Civil Infrastructure - 5th International Symposium on Life-Cycle Engineering, IALCCE 2016* (pp. 1285-1294). CRC Press / Balkema - Taylor & Francis Group.

IALCCE 2016: Fifth International Symposium on Life-Cycle Civil Engineering
October 16-19, 2016 / The Netherlands, Delft

Long-term performance of marine structures in The Netherlands - Validation of predictive models for chloride ingress.

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ABSTRACT: For many concrete infrastructural works a service life of 80, 100 or 200 years is required. To convince owners and authorities that these requirements can be met probability-based models for service life predictions have been developed. These models are based on theoretical and experimental laboratory studies. Many of these models focus on the probability of chloride-induced rebar corrosion. For a check of the reliability of one of these models, i.e. the DURACRETE model, predicted chloride profiles have been compared with chloride profiles measured in five marine concrete structures. Lessons learned from these existing structures in view of the reliability of numerical service life predictions are presented in this paper.

1 INTRODUCTION

Proper functioning of the infrastructure is vital for the nation’s economic stability and growth. Apart from its crucial role in the nation’s economy this infrastructure represents about 50% of its national wealth (Long. E.A. 2007). A substantial part of this infrastructure is built in concrete. Both the huge value of the infrastructure and its backbone function in proper functioning of the society, are two reasons for careful condition monitoring of the infrastructure over its total lifetime. Today large infrastructures are designed and built for a service life 100 to 200 years. Predicting a service life of this length is a big challenge. In the past service life predictions of concrete structures were largely based on experience and best practices. For large complicated structures, however, made with modern, unconventional types of concrete, experience-based rules and best practices will fail to provide reliable service life predictions. In those cases we have to refer to models. These models should be as simple and robust as possible, but at the same time as complete and consistent as possible. The completeness and consistency refers to all relevant factors involved in the degradation of a structure. These factors include the actions (loadings), materials properties, design details and execution. If all these factors are considered in a predictive model, the question still remains how reliable the predictions are. It is not realistic, of course, to wait for 100 to 200 years for a final judgment of the reliable of the predictions. In order to check the quality of service life predictions we can use existing structures for intermediate checks of the predictive models. In this paper it will be described how one of these models for service life prediction, i.e. the DURACRETE model (DuraCrete, 2000), performs by comparing predicted chloride profiles with those measured in five existing marine structures in The Netherlands.

2 DURACRETE MODEL FOR SERVICE LIFE PREDICTION

2.1 *Probabilistic approach*

The probabilistic concept is the basis for most of the currently used structural codes. More recently this

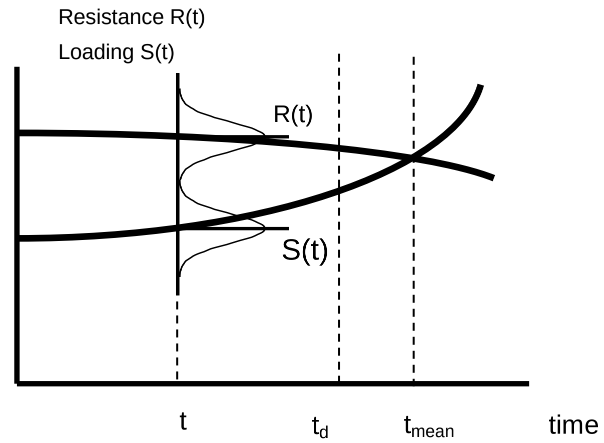

Figure 1. Schematic representation of probabilistic durability design. t_d is design service life.

concept has also been recommended for quantitative serviceability assessments (Richardson 2002). For a probabilistic assessment of the moment that a structure fails to meet the required performance criteria both the actions S(t) and the resistance R(t) can be considered variables with a normal distribution. Schematically the concept is shown in figure 1. Both the resistance and the loading may change with time. The resistance may increase due to continuing hydration and associated densification of the microstructure, but may decrease due to carbonation, rebar corrosion, micro-cracking or inherent materials ageing (Van Breugel 2014). The loads, or actions, may change as well, for example due to climate change.

2.2 *Basic concept of DURACRETE*

In the eighties of the past century the probabilistic approach has been adopted for development of the DURACRETE methodology for service life prediction of concrete structures (Siemes et al 1985). The concept was further developed in much more detail in the nineties in the European research project DURACRETE (DuraCrete 2000, Siemes et al 2000).

2.2.1 *Chloride ingress*

In DURACRETE the emphasis is on prediction of the onset of chloride-induced corrosion. Ingress of chloride ions is assumed to take place by diffusion. The chloride content at the steel C(x,t) is calculated with a transport formula based on Fick's second law of diffusion:

$$C(x,t) = C_s - (C_s - C_i)\, erf\left[\frac{x}{\sqrt{\{4\,k\,D(t)\,t\}}}\right] \qquad (1)$$

with C_s chloride surface content (% by mass of concrete or cement), C_i the initial chloride content (%), x the depth measured from the concrete surface (mm), D(t) the time-dependent diffusion coefficient (m^2/s), k an environmental coefficient (-). "*erf*" is the error function, stemming from solving Fick's second law of diffusion.

2.2.2 *Diffusion coefficient D(t) and ageing factor n*

In eq. (1) the diffusion coefficient D(t) is assumed to be time dependent. For D(t) Maage et al (1996) proposed:

$$D(t) = D_0\left(\frac{t_0}{t}\right)^n \qquad (2)$$

with D_0 the chloride diffusion coefficient at the reference time t_0, n the ageing factor (-) and t age of the concrete. The value of the ageing factor can be calculated from diffusion tests or the Rapid Chloride Migration tests (RCM).

The value of the ageing factor is crucial for the outcome of service life predictions. The big impact of the ageing factor is the reason why this coefficient is still heavily debated. With elapse of time the diffusion coefficient *decreases* due to progressive hydration. Microcracking, however, caused by autogenous and/or drying shrinkage, will result in an *increase* of the diffusion coefficient. The net result is, generally speaking, a decrease of the diffusion coefficient. This decrease is supposed to be appropriately described with the ageing factor n. In literature values for n are found from 0.25 to 0.6 for Portland cement concretes and from 0.7 to 0.9 for concretes containing fly-ash and/or slag (see Yu. Z. 2015). For mixtures with a water-binder ratio 0.45 Van der Wegen et al (2014) found n-values of 0.28 for blast furnace slag cement (CEM III/B), while for fly-ash blended cements n-values were determined in the range 0.89 to 1.02.

It is emphasized that discussions are still ongoing about the validity of Fick's second law of diffusion for simulating chloride ingress. Migration and convection may contribute to the chloride transport as well. Alternatively we could speak about chloride *penetration* or *effective diffusion* instead of chloride diffusion, leaving the actual transport mechanisms an open question. In this paper we still assume diffusion to be the dominating transport mechanism and consider Fick's second law sufficiently capable to simulate the chloride ingress.

2.2.3 *Critical chloride content*

The DURACRETE model involves a limit state formulation for initiation of chloride-induced corrosion. In a simplified form it says that failure, i.e. corrosion initiation, occurs when the chloride content C(x,t) at the reinforcement surface exceeds the critical (threshold) content C_{crit}. The critical chloride content is a complex function of concrete properties, in particular of the physics and chemistry (pH, water, oxygen, presence of voids) at the steel/concrete interface. Hence, there is not one single value for the critical chloride content that holds for all conditions. In the eighties Brown (1982) assumed negligible risk of corrosion for chloride contents at the steel surface $C<0.4\%$ (% weight of cement), possible corrosion for C=0.4-1.0%, probable corrosion for C=1.0-2.0% and certain corrosion for $C>2.0\%$. Angst (2011) pointed out that, since pitting corrosion is often the cause of failure, the size of the structures would be a relevant parameter in the risk of corrosion. This makes it even more complicated to define a single value of a critical chloride content.

Today there is at least general agreement about the fact that the probability of corrosion increases with increasing chloride content (Gaal 2004, Vassie 1984). For real structures (as opposed to laboratory specimens) a value of 0.5% chloride ion by mass of cement is considered to be the best mean value for

Portland cement concrete. No well-established value for Blast Furnace Slag cement is available yet.

2.4 *Application of the DURACRETE model*

The DURACRETE model, i.e. eq. (1), can be applied in the design stage of new structures to calculate the time when the chloride content C(x,t) reaches the threshold value C_{crit} by inserting values for the cover depth, the expected surface chloride content (based on experience, e.g. from DURACRETE tables) and chloride diffusion coefficients experimentally determined from trial concrete mixtures. The DURACRETE method includes allowance for the scatter and distribution type of the input variables and the required reliability (or probability of failure) of the result. Taking into account the stochastic character of the variables, full probabilistic and semi-probabilistic calculations are possible, either using statistical parameters for all variables or by using (partial) safety factors.

The indicated methodology used for new structures can also be used for assessment of existing structures (DuraCrete 2002). Applying the model for existing structures, however, is relatively new yet. For existing structures some of the input parameters are different or even not available. For example, it is not possible to measure the diffusion coefficient D_0, i.e. the diffusion coefficient at 28 day, of concrete that is already 20 years old. On the other hand, the chloride surface content and the cover depth and their statistical distribution are now exactly known from measurement on the site. Moreover, the chloride profiles can be measured and can be used for back-calculation of the diffusion coefficient D(t) and the ageing factor n. In this way the predictive model can ‘learn’ from the performance of existing structures, resulting in improvement of the accuracy of future service life predictions.

3 MODEL VALIDATION THROUGH FIELD INVESTIGATIONS

3.1 *Description of structures*

Six structures were selected for field investigations. These structures were thought representative for a larger group of marine structures. Criteria were age, cement type, production method (cast in situ or prefabricated) and accessibility. Some characteristics of the structures are described in Table 1. Of each of these structures one to six test areas were defined and investigated in detail.

The Pier at Scheveningen is a bridge-type structure, with a promenade deck composed of precast cross beams and precast slabs, supported by precast piles (not investigated). Some parts of the deck were cast in situ. Marine exposure due to waves splashing occurs on the bottom side of the deck, which is between 5 and 11 m above mean sea level. Two test areas were located on slabs, two on beams and two on the cast in situ deck.

Table 1: Characteristics of the investigated structures

Structure	Year of construction	Cement type	Production
Pier Scheveningen	1960	OPC	Precast
Sluice Haringvliet	1960	BFSC	In situ
Quay wall Calandcanal	1968	BFSC	In situ
Quay wall Hartelhaven	1973	BFSC	In situ
Quay wall Europahaven	1982	BFSC	In situ
Eastern Scheldt storm surge barrier	1984	BFSC	In situ/ Precast

The Haringvliet "spuisluizen" is a river discharge complex, composed of cast in situ piers and precast bridge elements. The piers reach from sea level up to +14 m height. All test areas were facing the North Sea, one located just above sea level, two at +9 m and one on +14 m above sea level.

Three quay walls were investigated, all located in Rotterdam harbour. They are box girder type structures, supported by precast piles. In each quay wall, one test area was located on the vertical wall facing the seaside.

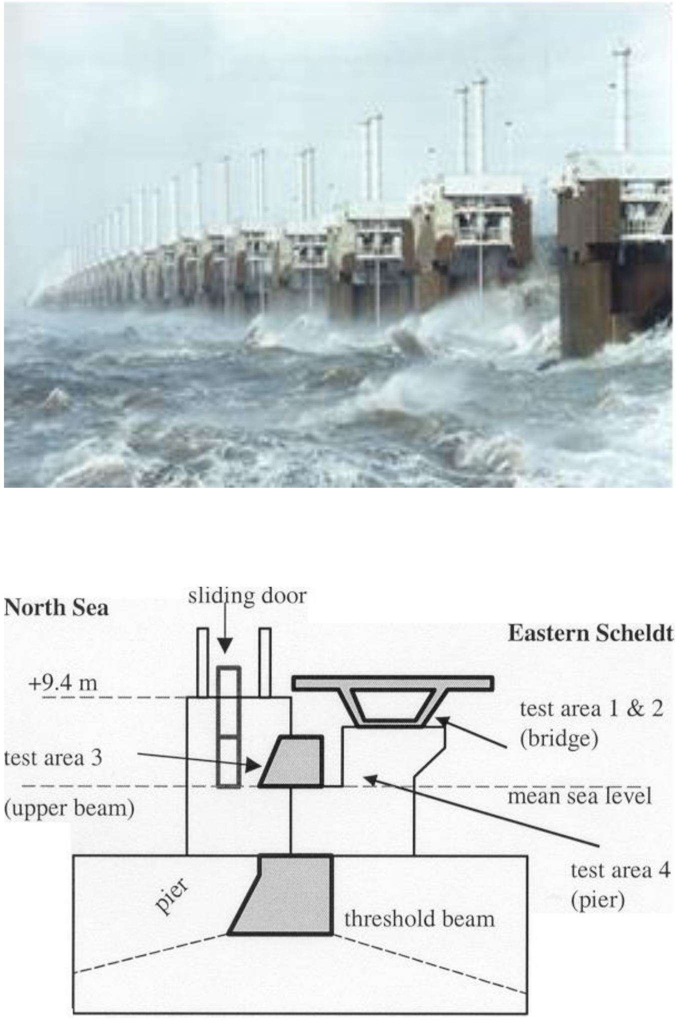

Figure 2. Eastern Scheldt Storm Surge Barrier (top) with cross section and indication of test locations (bottom).

The Eastern Scheldt Storm Surge Barrier (Figure 2), built in the 1970s, consists of 65 piers cast in a dock nearby. The piers were horizontally connected by threshold beams on the sea bed and upper beams extending from sea level to about +8 m. Beams were cast in a plant nearby as elements of about 20 m length, two of which were cast together on site to form one span. Bridge elements are spanning the tops of the piers, which were precast in halves like the beams. The main structure was designed for a service life of 200 years, whereas the bridge superstructure was designed for 50 years. On this structure test areas were located (Figure 2) on one pier (facing South), just above sea level; on an upper beam (facing West), at about +4 m; and on the Eastward facing lower part of a bridge element (at +9 m), relatively sheltered by an overhanging part of the driveway.

3.2 *Test areas and methods*

In the test areas of about one by one meter all reinforcement was located using a scanning cover depth meter. Although emphasis was on the analysis of the chloride profiles, several additional checks of the condition of the concrete were done. The concrete surface was inspected visually. Steel potentials were measured using a silver-silver chloride reference electrode. Concrete resistivity was measured using a four-point (Wenner type) probe (Polder 2000). Carbonation depths were measured by spraying phenolphthalein in freshly made holes. Cores were taken for chloride profile analysis (six cores of ∅50 mm per test area), for polarizing and fluorescence microscopy (Polder et al. 1995) (three of ∅50 mm) and for other tests (six of ∅100 mm), including strength testing. Chloride content of the concrete was determined using Volhard's titration method. Chloride was expressed as percentage of cement mass, assuming that all acid soluble mass was hardened cement paste (including 18% hydration water). This is justified, as virtually all aggregate was siliceous. The initial chloride content of the concrete, C_i, was assumed to be 0.01% chloride by mass of cement, except for Haringvliet. In the latter structure it appeared to be about 0.1%.

The measured chloride profiles were used the determine the value of the diffusion coefficient D(t) and the ageing factor n by back-calculation (fitting). In most cases the first data point, i.e. in the outer layer of the concrete cover, was ignored in the fitting procedure, especially when it was considerably lower than the second data point. This because the chloride content in the outer zone of up to about 10 mm, is strongly affected by convection/absorption, and less by diffusion. A similar procedure of ignoring the chloride data in a thin surface zone has also been recommended by Andrade et al (2014).

4 ANALYSIS OF EXPERIMENTAL DATA

4.1 *Summary of results*

In this section some of the results of the field investigation are summarized. For more details reference is made to (de Rooij et al 2005).

Visual inspection generally showed no major defects, mainly some marine growth on and light erosion of the concrete surface. Mechanical damage was observed in a few of the quay walls.

Corrosion related damage (rust staining, cracking, spalling) was observed in parts of the precast slabs and in a part of the cast-in-situ concrete of Pier Scheveningen. In both areas, old repairs were present where corrosion and cracking had earlier reappeared already.

It was found that about 25% of the *precast* slabs of the Pier Scheveningen had minimum cover depths of 20 to 25 mm. At these locations corrosion related damage had occurred. In slabs where the cover depth was 30 to 35 mm no visible corrosion was observed. The *cast-in-situ* deck, with about 35 mm cover depth, showed extensive corrosion damage (cracking, spalling and ineffective repairs). None of the other structures showed visual signs of corrosion.

Carbonation depths were low in all cases, typically about 2 mm with occasional values of 5 mm.

Concrete compressive strengths ranged from 50 to 75 MPa for blast furnace slag cement concrete and were about 120 MPa for Portland cement concrete (precast beams Pier Scheveningen).

Polarizing and fluorescence microscopy (PFM) showed that all concrete was made well and quite homogeneous, with good mixing of raw materials, compaction and curing. All samples had been made with Blast Furnace Slag cement with a high percentage of slag (>65%), except for precast concrete of Pier Scheveningen, which was made using Ordinary Portland cement.

The apparent water-to-cement ratios inferred from comparison of the capillary porosity with samples from a reference collection were low, generally 0.45 or less. In some cases, historic documentation suggested that a w/c of 0.55 had been used.

4.2 *Cover depth*

Cover depths observed in the test areas are summarized in Table 2 by their mean value and standard deviation. It appears that cover depths of structures made in the 1960s show a large variation: Haringvliet has very high (mean) values up to 90 mm, while Pier Scheveningen had low values of about 25 mm. At Scheveningen, extended measurements of precast slabs (over a much larger surface than the two test areas) showed that cover depths had a bimodal distribution with peaks at 25 mm and 35 mm. The original design value was probably 35 mm, the

Table 2. Cover depth at test areas of investigated structures

Structure	Test area	Cover depth [mm] Mean	Cover depth [mm] Stand. dev.
Pier Scheveningen	Precast slab (+7m)	26	9
	Cross beam (+5m)	42.4	5.0
	In situ beam (+5m)	36.5	1.7
Sluice Haringvliet	Pier 11, low (+1m)	71.1	4.9
	Pier 11, middle (+9m)	79.5	4.1
	Pier 11, high (+14m)	90.1	5.1
Q.W. Calandkanaal	Quay wall (+1m)	42.2	4.0
Q.W. Hartelhaven	Quay wall (+1m)	54.6	6.6
Q.W. Europahaven	Quay wall (+1m)	56.2	3.9
Eastern Scheldt	Pier Hammen 9 (+1m)	57.5	6.0
storm surge barrier	Upper Beam (+4m)	69.1	2.5
	Bridge element (+9m)	41.1	1.4

lower values are related to deviations in the production process. The cover depths of the Rotterdam quay walls suggest an increase over the years from about 40 mm (Caland, 1968) to 55 mm (Hartel, 1973 and Europa, 1982), indicating an increasing awareness of the importance of the cover depth. The specifications required 40 mm for the quay wall in the Hartelhaven and 50 mm for the quay wall in the Europahaven. No data could be found about the quay wall at the Calandkanaal.

4.3 *Chloride profiles*

As an example, Figure 3 presents the measured chloride profiles (mean values and mean plus and minus the standard deviation), the best fitting diffusion profile (neglecting the first data point closest to the exposed surface), and the DURACRETE prediction with equation (1) for the test area on the upper beam of the Eastern Scheldt Barrier (age about 18 years). The figure shows that in the zone 0-10 mm strong differences exist between measured, fitted and predicted chloride profiles. Beyond 10 mm depth, however, they are all quite close. The 'best fit' curve coincides with the mean measured curve from 15 mm (10-20 mm) on. The DURACRETE prediction is a bit lower than the mean measured curve, close to the 'mean minus standard deviation' curve.

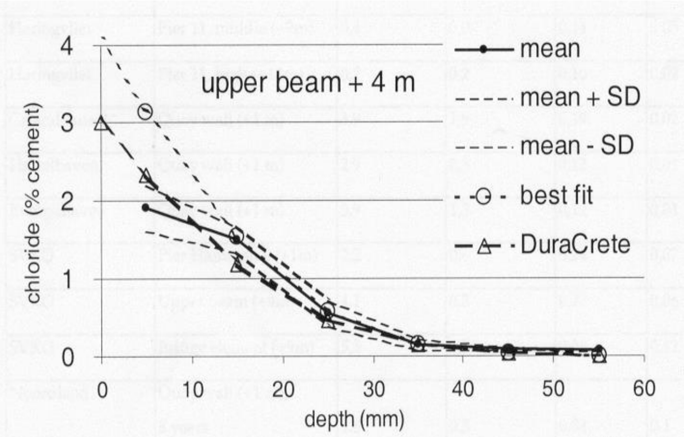

Figure 3. Chloride profiles from the test area on the upper beam of the Eastern Scheldt Barrier (after 18 years). Mean of six measured profiles, mean plus and mean minus standard deviation, best fit and prediction with DURACRETE (eq. (1)).

In spite of the fact that the test areas were relatively small (1×1 m^2) and the good material homogeneity as observed by microscopy, the scatter in the test results was rather large. Obviously, only microscopic observations are not able to discern differences in the resistance of the concrete for chloride ions to penetrate into the concrete. Or, the microclimate differs more than assumed on the basis of global observations.

4.4 *Surface chloride content*

Most of the observed surface chloride contents are in the usual range of 2 to 5%. Literature data on similar concrete compositions provide surface contents in this range for marine splash zone exposure and for submerged exposure in the laboratory (Polder 1996). In the present set of data, Haringvliet middle and high surface contents are much lower. This suggests that those two test areas do not belong to the splash zone group.

4.5 *Effective Cl-penetration coefficient – Effect of moisture content*

Effective diffusion coefficients, D(t), were obtained from curve fitting of measurement chloride profiles. The results are summarized in Table 3. It appears that the present diffusion coefficients for OPC concrete are, on average, a bit higher ($0.14.10^{-12}$ to $0.33.10^{-12}$ m^2/s) that those for BFSC concrete ($0.10\ 10^{-12}$ to $0.28.10^{-12}$ m^2/s). Moreover, the scatter in the values for OPC concrete is higher than for BFSC concrete.

Particularly the D(t) values for OPC concrete are substantially lower than those found in earlier studies of Polder & Larbi (1995). In the latter studies apparent diffusion coefficients were calculated from chloride profiles in concrete samples submersed in sea water at a depth of 5 to 100 m for 16 years. In those studies OPC concrete with w/c 0.40 showed

Table 3. Diffusion coefficients D(t) of investigated structures

Structure	Test area	Diffusion coeff. D(t) $*10^{-12}$ m^2/s Mean	Diffusion coeff. D(t) $*10^{-12}$ m^2/s Stand.dev.
Pier Scheveningen	Precast slab (+7m)	0.14	0.03
	Cross beam (+5m)	0.28	0.14
	In situ beam (+5m)	0.33	0.14
Sluice Haringvliet	Pier 11, low (+1m)	0.12	0.04
	Pier 11, middle (+9m)	0.14	0.05
	Pier 11, high (+14m)	0.10	0.02
Q.W. Calandkanaal	Quay wall (+1m)	0.19	0.02
Q.W. Hartelhaven	Quay wall (+1m)	0.12	0.01
Q.W. Europahaven	Quay wall (+1m)	0.12	0.01
Eastern Scheldt	Pier Hammen 9 (+1m)	0.24	0.07
storm surge barrier	Upper Beam (+4m)	0.27	0.06
	Bridge element (+9m)	0.28	0.12

D(t) values of 1 to 3.10^{-12} m^2/s and BFSC concrete with w/c 0.42 about 0.3 10^{-12} m^2/s. Neglecting minor differences of water-cement ratio and age, this suggests that Portland cement concrete in the splash zone, considered in the present study, has a much lower effective diffusion coefficient than in submerged exposure. An explanation for this finding could be the following. Polder et al (2005) performed tentative analysis, suggesting that drying out of Portland cement concrete, in the higher marine splash zone, may have slowed down chloride transport considerably as compared to transport in water saturated concrete. Eventually, effective diffusion in Portland cement concrete may be about as slow as in (wet) Blast Furnace Slag cement concrete. Experiments on saturated concrete in the laboratory and from natural submersion tests until now have indicated that chloride diffusion in slag cement is much slower than in Portland cement based materials (Page et al. 1981).

The influence of drying out on chloride transport in concrete has also been investigated under equilibrium non-saturated conditions (Vera et al. 2002) and under simulated wetting and drying cycles with salt solution (Polder et al. 2002). Drying out of the concrete, increased concrete resistivity and reduced transport rates were found to be related (Polder et al 2002). The effects of drying of concrete on chloride transport, in particular in the splash zone, however, require more study.

From an analysis of chloride profiles in 100 years old concrete Andrade et al. (2014) calculated an apparent diffusion coefficient of 2-4 10^{-12} m/s. These values are in good agreement with those found by Polder et al (1995) in the submersed 16 year old OPC concrete. The diffusion coefficients determined by Andrade were obtained from a 100 year old structure. At that time the cement used was most probably Portland cement.

5 PREDICTABILITY OF SERVICE LIFE

In Figure 3 an illustration is given of the predictability of the chloride profile with the DURACRETE model. In this example the theoretical curve and measurement points fit quite well, except for the outer surface layer of the structure. More good fits could be shown, but also discrepancies of theory and practice was found. Improvement of the reliability of the predictions could be obtained by adjusting the formulation for the time-dependency of the penetration coefficient D(t), i.e. the ageing factor n in eq. (2), and the environmental factor k in eq. (1).

It was found that for the type of structures and exposure conditions considered in marine environments encountered at the North Sea the factor k can be replaced by an Arrhenius-type factor k(T), allowing for the mean temperature T_e during the year, viz.:

$$k(T) = e^{\frac{E}{R}\left(\frac{1}{T_{ref}} - \frac{1}{T_e}\right)} \quad (4)$$

with E the activation energy [J/mol], R the universal gas constant [8.136 J/mol K], T_{ref} the reference temperature (293 K) and T_e the annual mean air temperature (K). For an activation energy of E = 40,000 J/mol and an annual mean temperature of about 10°C, a value of k(T) = 0.55 is found.

For the ageing factor n it was found that good results could be obtained with a value of about 0.48 with a standard deviation 0.07. It is interesting to note that Gehlen found a similar value of 0.45 for slag cement from literature data for profiles taken after up to 60 years (Gehlen 2000). From analysis of 100 years old concrete Andrade et al. (2014) calculated n-values between 0.48 to 0.875.

With the adjustments of the two DURACRETE factors k and n, most of the chloride profiles could be described fairly accurately. Assuming a chloride threshold factor for initiation of corrosion of 0.5 % chloride by mass of cement, it was found that for the investigated structures with the given depth of the concrete cover the risk of corrosion was negligible. This corresponded well with the observations. Only in parts of the Pier Scheveningen, were the concrete cover was much less than specified and a Portland cement was used with a relatively high w/c, serious corrosion was observed. The model did not predict otherwise.

6 CONCLUSIONS AND OUTLOOK

Today, for important infrastructural works a required design lifetime of 100 to 150 years is not exceptional. With the knowledge of today it is too big a claim to state that we can rely completely on predictive durability models. The experience with currently used predictive models is limited to about 25 year, which is too short to claim very accurate predictions for 100 to 200 year service life (Andrade et al. 2014). From the best predictive models currently available we can learn, however, that for a set of assumed boundary conditions a set of assumed loadings will result in a certain degree of deterioration of a structure. The reliability of these predictions depends on the *completeness* and *consistency* of the models and on the *quality* of the input, i.e. the assumed boundary conditions and loadings.

With increasing ‘maturity’ of predictive models the length of the period over which reliable predictions are possible will increase. In this paper it has been shown that numerical models, like DURACRETE model, can describe chloride profiles sufficiently accurate to support decision making processes concerning required density of the concrete and thickness of the concrete cover in order to ensure a predefined service life of marine structures exposed to moderate climate conditions. In order to reach that degree of reliability the model was validated by comparing model predictions with observations at six marine structures at the North Sea. By adjusting only two model parameters, i.e. the environmental factor k and the ageing factor n, a good correlation was established between predictions and chloride profiles observed in the examined structures. The adjustments were still within the range of values currently adopted for these parameters. This points to a high degree of robustness of the model, even though there is still room for refinement of the model. Further refinement of the model, however, should keep step with the accuracy with which other durability-affecting factors are under control. In this respect it is considered of utmost importance to implement a consistent set of procedures focusing on producing the specified quality of the concrete and depth of the concrete cover (Van Breugel 2005, 2008). In a consistent set of procedures predictive models, like the DURACRETE model, are considered strong tools. Firstly, for supporting decision making processes in the design stage of a project and, secondly, for educational purposes.

It is emphasized that higher accuracy and reliability of models for service life prediction is not only a matter of serving owners with better information about the condition of their assets. With the increasing awareness of the high impact of building activities on the environment improved condition monitoring should be in the interest of the society as a whole. In the end better models should enable us to mitigate the environmental burden of building activities, thus contributing to a more sustainable society.

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