# Durability of marine concrete structures – field investigations and modelling

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**This article presents a series of investigations on six concrete structures along the North Sea coast in The Netherlands. They had ages between 18 and 41 years and most of them were made using Blast Furnace Slag cement. Visual inspections showed corrosion damage in only one structure, related to relatively low cover depths. All structures showed considerable chloride ingress with a large scatter within the relatively small tested areas. The interpretation was based on the DuraCrete model for chloride ingress. Curve fitting to chloride profiles produced chloride surface contents and apparent diffusion coefficients. Comparison was made to previously published data on chloride ingress and electrical resistivity of similar concretes. It was found that a single mean value and standard deviation applied to all concrete up to 7 m above mean sea level for the chloride surface content. Above 7 m, the local microclimate had a decisive influence, either increasing or reducing the chloride surface content. Apparent chloride diffusion coefficients did not depend on height above sea level. Their age dependency was expressed in a single value for the exponential aging coefficient. A simplified environmental factor was adopted from literature. A probabilistic model for corrosion initiation in Blast Furnace Slag Cement concrete in marine environment was proposed, the DuMaCon version of the DuraCrete model. Its application is explained for design of new structures and for assessment of existing structures. Issues for further research are the critical chloride content and the target failure probability for corrosion initiation, the effect of drying out on chloride transport in the marine splash zone and the nature and influence of spatial variation of chloride ingress.**

*Key words: Concrete, marine environment, chloride, blast furnace slag cement, reinforcement corrosion, service life, probabilistic model*

## 1 Introduction

Durability of Concrete structures in marine environment has been an issue for many decades, due to the perception of sea water as aggressive to concrete and reinforcement and the long service life that is expected for marine infrastructure such as harbour and coastal defence structures. In particular in the 1970s, a lot of work has been done due to increasing construction for the oil and gas offshore industry, e.g., in Concrete in the Oceans [Leeming 1989]. In The

Netherlands, a collective research programme was carried out under supervision of CUR committee B23, resulting in [CUR 100, 1981]. In that study, sixty structures of various ages were visually inspected and five were investigated in more depth. Generally the durability was found to be "good"; relatively little deterioration was observed. The most important threat to durability was found to be corrosion of reinforcement due to chloride ingress, mainly in older structures with relatively low concrete cover to the reinforcement [Wiebenga 1980]. In view of the young age of the investigated structures relative to the slow rate of degradation, it was recommended to carry out a similar study in about 15 years time.

In the 1990's, a group of European researchers developed a methodology for quantitative service life design of concrete structures, entitled "DuraCrete", which was based on the approach proposed in the 1980's [Siemes et al. 1983]. Explanations of the principles and examples of its application have been published [CEB 1997, Siemes et al. 1998]. DuraCrete's final report includes models for predicting corrosion initiation due to chloride ingress and due to carbonation as well as models for propagation of corrosion and subsequent cracking and spalling [DuraCrete R17, 2000]. Using the DuraCrete methodology it is possible to quantify the reliability of a structure with respect to predefined limit states that concern durability [Vrouwenvelder & Schiessl 1999].

This new quantitative approach and the availability of new investigative techniques such as electrochemical methods and concrete microscopy prompted CUR and TNO to start an investigation into durability in marine environment in 2000 under the title "Durability of Marine Concrete structures" (DuMaCon). Its objectives were to quantify the durability of marine structures in The Netherlands and to provide degradation models and associated failure probabilities for existing marine structures. This article describes the original DuraCrete model, the fieldwork carried out to collect data, an overview of the results, some comparison to data from other sources, subsequently proposed modifications to the DuraCrete model and recommendations for its application.

## 2 DuraCrete model for corrosion initiation due to chloride ingress

### *2.1 **Basic concept***

The DuraCrete degradation model for chloride induced corrosion is based on the concept of chloride transport into concrete by diffusion and initiation of reinforcement corrosion when a critical chloride content is exceeded at the steel surface. Diffusion modelling of chloride ingress into concrete was proposed in the 1970s [Collepardi et al. 1972] and further developed in the following decades [e.g. Bamforth & Price, 1993, Maage et al. 1996]. After the critical "threshold" chloride content has reached the steel and has broken down its normal passivation, the steel starts to dissolve. Dissolved iron ions react to form corrosion products, at some point in time causing expansive stresses and cracking of the concrete cover. Eventually, the loss of steel cross section may become critical with respect to structural capacity [Vrouwenvelder & Schiessl 1999]. The two-stage concept of initiation and propagation of corrosion was developed by [Bazant 1979 and Tuutti 1982]. In most service life design approaches, however, the initiation period is considered dominant and the propagation period is neglected. For more information on corrosion of steel in concrete the reader is referred to [Bertolini et al. 2004].

### 2.2 *The DuraCrete model*

The DuraCrete model involves a limit state formulation for chloride induced corrosion initiation which can be simplified by stating that failure (that is, corrosion initiation) occurs when $C > C_{crit}$, with C the chloride content at the reinforcement surface and $C_{crit}$ the critical (threshold) content. 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. Nowadays it is realised that there is no single general value for it, but rather a gradual increase of the probability of corrosion with increasing chloride content [Vassie 1984, Gaal 2004]. 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. Further treatment of this subject is outside the scope of this article.

According to the DuraCrete model for chloride transport, the chloride content at the steel C(x.t) is a time dependent function described by:

$$C(x,t) = C_s - (C_s - C_i)\, erf\left( \frac{x}{2\sqrt{K D_0 t \left(\frac{t_0}{t}\right)^{n_{Cl}}}} \right) \qquad (1)$$

With $C_s$ chloride surface content (% by mass of concrete or cement); $C_i$ initial chloride content (%); x depth of the steel (m); $D_0$ diffusion coefficient ($m^2/s$) at $t_0$ (s, usually 28 days); K environmental coefficient (-); $n_{Cl}$ ageing exponent (-); *erf* errror function, stemming from solving Fick's second law of diffusion.

### 2.3 *Application of the model*

The DuraCrete model can be applied in the design stage of new structures by inserting into equation (1) values for the design cover depth, the expected surface content (based on experience, e.g. from DuraCrete tables) and experimentally determined chloride diffusion coefficients of trial concrete mixes, to calculate the point in time when C(x,t) reaches $C_{crit}$, whose value also is taken from experience or tables. The input values for cover depth and chloride diffusion coefficient are varied until the calculation indicates that corrosion initiation is postponed until the end of the desired service life period. In a later stage, concrete producers must prove that their product can meet the required diffusion coefficient. However, this type of calculation is deterministic and consequently produces the mean time-to-corrosion initiation. The DuraCrete service life design method includes taking into account the scatter and distribution type of the input variables and the required reliability (or probability of failure) of the result. Generally, the accepted probability of failure at the end of the service life will be a few percent. 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. Various DuraCrete reports provide all such information. In this paper, we will not go into detail of these probabilistic calculations.

In the past few years, (semi-)probabilistic calculations have been made taking into account that exceeding the limit state “corrosion initiation” is a serviceability limit state (SLS), with an associated target failure probability of a few percent (reliability index $\beta$=1.8). This approach has been used for the service life design of the Westerschelde Tunnel [Siemes et al. 1998, Gehlen 2000], the Groene Hart Tunnel and other structures in the High Speed railway Line (HSL) in The Netherlands.

DuraCrete explicitly states that the same methodology can be used for assessment of existing structures [DuraCrete R17, 2000]. Application of the model to practical cases, however, is relatively new. For existing structures some of the input parameters are different. For example, one of the input parameters for the model is the measured value of the diffusion coefficient of 28-days old concrete. It is not possible to measure this value on concrete that is already 20 years old. On the other hand, the chloride surface content and the cover depth and their statistical distribution can be established experimentally; hence their values can be assessed experimentally and do not have to be assumed like in the design stage.

The DuMaCon study aimed to collect data on existing marine structures, carry out model calculations and to validate and/or modify the model.

## 3 Field investigations

### *3.1* ***Structures***

Six structures were selected for field investigations that were thought representative for a larger group of structures. Criteria were age, cement type, production (cast in situ or prefabricated), availability and interest of the owner in the study. Some characteristics of the structures are described in Table 1. On each of these structures, one to six test areas were investigated in detail.

*Table 1: Characteristics of the investigated structures*

| **Structure** | **Year of construction** | **Cement type** | **Production** |
|---|---|---|---|
| Pier Scheveningen | 1960 | Portland cement, Blast furnace slag cement | Precast<br>Cast in situ |
| Discharge sluice Haringvliet | 1960 | Blast furnace slag cement | Cast in situ |
| Quay wall Calandkanaal | 1968 | Blast furnace slag cement | Cast in situ |
| Quay wall Hartelhaven | 1973 | Blast furnace slag cement | Cast in situ |
| Quay wall Europahaven | 1982 | Blast furnace slag cement | Cast in situ |
| Eastern Scheldt Storm Surge Barrier | 1980-1984 | Blast furnace slag cement | Precast in field plant, cast in situ |

All structures are located on the coast or the estuaries and harbours of the Southwestern part of The Netherlands. Historical records were studied in preparation of the inspections. The amount of information available regarding the composition and production of concrete was quite low. In most cases, relevant information such as cement content, water-to-cement ratio and curing was not well documented.

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), with some parts of the deck cast in situ. Marine exposure due to waves splashing occurs on the underside of the deck, which is between 5 and 11 m above mean sea level; the top surface of the deck is protected by a (later) building and was not investigated. Two test areas were located on slabs, two on beams and two on the cast in situ deck. A schematic is shown in Figure 1.

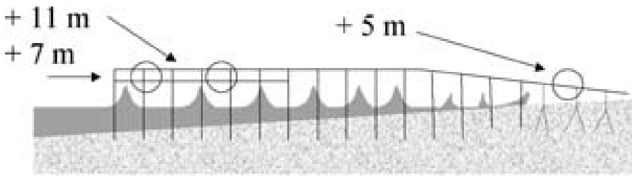


*Figure 1 Schematic of Pier Scheveningen; circles indicate test areas on underside of deck; top surface of deck is protected*

The Haringvliet “spuisluizen” is a river discharge complex, composed of cast in situ piers and precast bridge elements (not investigated). The piers reach from sea level up to + 14 m height. All test areas were (vertically) facing the North Sea, one located just above sea level, two at + 9 m and one on +14 m above sea level. A schematic of their location is shown in Figure 2.The three quay walls are 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 at the high tide level (+ 1 m).

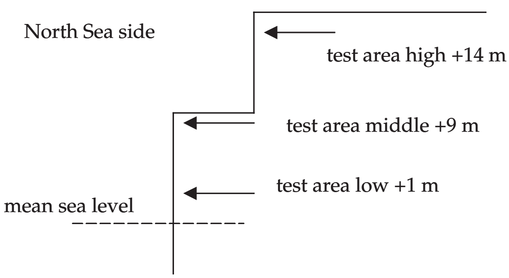


*Figure 2 Schematic of a pier of Haringvliet spuisluizen (discharge complex)*

The Eastern Scheldt Storm Surge Barrier was designed in the 1970s to protect the low-lying hinterland against flooding. The main structure consists of 65 piers that were cast in a dock nearby, 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. Steel sliding doors are lowered between the piers to close the estuary in times of high flood risk; in the raised position they allow salt-water movement under normal conditions. 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 using then existing service life prediction models [Hageman 1982]; the bridge superstructure was designed for 50 years. On this structure, see Figure 3, three test areas were located on one pier: test area 1 on the Eastward facing lower part of a bridge element (at + 9 m), relatively sheltered by an overhanging part of the driveway, test area 3 on an upper beam (facing West), at about + 4 m and test area 4 on one pier (facing South), just above sea level. Test area 2 was situated on a bridge element composed of lightweight concrete. Furthermore, additional information was obtained from cores taken in the pier below water level and from a large number of cores taken from bridge elements in other spans across the complete barrier. In addition, cores were taken from quay wall Noordland built near the barrier in 1984, from which previously cores were taken and analysed for chloride ingress in 1992 [Polder et al. 1995].

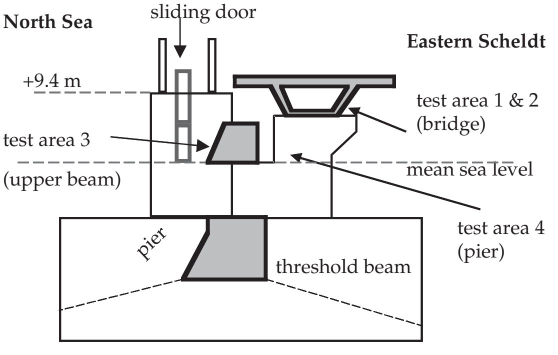


*Figure 3 Side view of Eastern Scheldt Storm Surge Barrier Pier with test areas indicated; NB test area 2 is in lightweight concrete in another span than the other three test areas*

*3.2* ***Test areas and methods***

Each test area had dimensions of about one by one metre. In that area, all reinforcement was located using a scanning cover depth meter and 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 $\phi$50 mm per test area), for polarizing and fluorescence microscopy [Polder & Larbi 1995] (three of $\phi$50 mm) and for other tests (six of $\phi$ 100 mm), including strength testing. Cores for chloride analysis were cut in the laboratory by diamond sawing in slices of about 10 mm thickness, which were dried, crushed and dissolved in hot nitric acid. Chloride was determined in the liquid using Volhard's titration. 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 resulting chloride profiles are fitted to obtain an effective diffusion coefficient, using a commonly used least squares procedure based on the solution of Fick’s second law of diffusion:

$$C(x,t) = C_s - (C_s - C_i)\, erf\left(\frac{x}{2\ \sqrt{D_t\, t}}\right) \qquad (2),$$

with $C_s$ chloride surface content (%); $C_i$ initial chloride content (%) ; x depth (m); $D_t$ effective diffusion coefficient ($m^2/s$) at the time t of inspection (s). In most cases the first data point was ignored in the fitting procedure, especially when it was considerably lower than the second data point. The initial chloride content of the concrete was assumed to be 0.01% chloride by mass of cement, except for Haringvliet, where it appeared to be about 0.1%. Reasoning back with builders from the structure resulted in the assumption that storage of concrete aggregates and sand near the building location would be the most likely cause of this elevated initial chloride content in the concrete of Haringvliet.

# 4 Analysis of data

### *4.1 **Summary of results***

The results are summarized here; for more details reference is made to [Rooij & Polder, 2005]. Visual inspection generally showed no major defects, mainly some marine growth on and light erosion of the concrete surface; mechanical damage was present in a few of the quay walls. Corrosion related damage (rust staining, cracking, spalling) was observed in part 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 reappeared. It appeared that about 25% of the precast slabs had minimum cover depths of 20 to 25 mm, which was where corrosion related damage had occurred; slabs without visible corrosion had 30 to 35 mm cover depth. The cast in situ deck with about 35 mm cover depth also 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 to samples from our reference collection were low, generally 0.45 or less. In some cases, historical documentation suggested that a w/c of 0.55 had been used. It appears that hydration, in particular of slag particles, of concrete exposed to marine environment over prolonged time is able to cause a substantial reduction of the capillary porosity.

In some concrete, the records suggested that ground trass had been used. Microscopically this could not be established [Nijland et al. 2005].

### 4.2 *Cover depths*

Cover depths observed in the test areas are summarized in Table 2 by their mean 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 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). The specifications required 40 mm for Hartel and 50 mm for Europa (Caland unknown).

*Table 2 Summary of cover depths for test areas (m  mean and s standard deviation)*

| **Structure** | **Test area** | **Cover depth (mm)** | |
|---|---|---|---|
| | | m | s |
| Pier Scheveningen | Precast slab (+ 7 m) | 26 * | 9 * |
| Pier Scheveningen | Cross beam (+5 m) | 42.4 | 5.0 |
| Pier Scheveningen | In situ (+ 5 m) | 36.5 | 1.7 |
| Haringvliet | Pier 11, low (+ 1 m) | 71.1 | 4.9 |
| Haringvliet | Pier 11, middle (+9m) | 79.5 | 4.1 |
| Haringvliet | Pier 11, high (+14m) | 90.1 | 5.1 |
| Calandkanaal | Quay wall (+1 m) | 42.2 | 4.0 |
| Hartelhaven | Quay wall (+1 m) | 54.6 | 6.6 |
| Europahaven | Quay wall (+1 m) | 56.2 | 3.9 |
| Eastern Scheldt Barrier | Pier Hammen 8 (+1m) | 57.5 | 6.0 |
| Eastern Scheldt Barrier | Upper beam (+4m) | 69.1 | 2.5 |
| Eastern Scheldt Barrier | Bridge element (+9m) | 41.1 | 1.4 |

** Note: Test area in low end of bimodal distribution; see text.*

*4.3* ***Chloride profiles***

As an example, Figure 4 presents the measured chloride profiles (mean of six; mean plus and mean minus standard deviation), the best fitting diffusion profile using equation (2), neglecting the first data point, and the DuraCrete prediction from equation (1) for the test area on the upper beam of the Eastern Scheldt Barrier (age c. 18 years). For input parameters for the DuraCrete prediction see Table 3. The Figure shows that in the zone 0 – 10 mm strong differences exist between measured, fitted and predicted chloride profiles; however, beyond 10 mm depth 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. Actually in this example the scatter is relatively low; in most cases, much larger scatter was present. Mean and standard deviations for diffusion coefficients and chloride surface contents from the best fitting curves for the test areas (using equation (2)) are presented in Table 4. Means of profiles from quay wall Noordland have been added.

*Table 3 Input parameters DuraCrete prediction for upper beam of Eastern Scheldt barrier (see Figure 4)*

| **Parameter** | **Source** | **Value** |
|---|---|---|
| Environmental zone | | Splash zone |
| Cement type | Inspection | Blast furnace slag cement |
| Water cement ratio | Inspection | 0.45 |
| Curing | Assumption | 1 day |
| A factor | [DuraCrete R17, 2000] | 6.77 |
| $k_c$ factor | [DuraCrete R17, 2000] | 2.40 |
| $k_e$ factor | [DuraCrete R9, 2000] | 0.77 |
| $D_{0,\ RCM}$ | TNO Database | $4.5 \times 10^{-12}$ m$^2$/s |
| Age at inspection | Inspection | 16 year |
| $n$ exponent | [DuraCrete R17, 2000] | 0.60 |

Chloride ingress was found to be subject to considerable scatter. Taking into account the small size of the test areas (c. 1 x 1 m$^2$, suggesting homogenous exposure) and the good material homogeneity as observed by microscopy, no clear explanations are available. Possibly variations of the microstructure or in the microclimate are overlooked. For the moment, chloride ingress must be considered a stochastic process (as is corrosion initiation, for that matter [see e.g. Vassie 1984]). In addition, the spatial variability on macro scale (across a complete structure) is not well understood [Li, 2004].

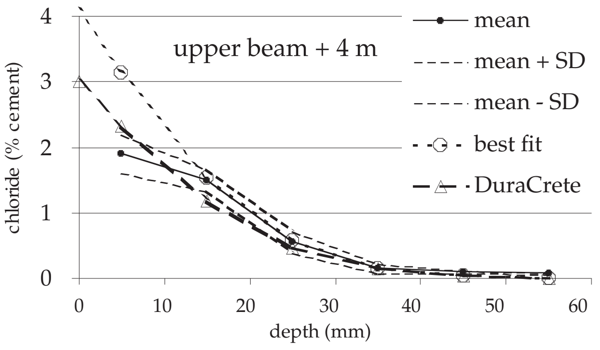


*Figure 4 Chloride profiles from the test area on the upper beam of the Eastern Scheldt Barrier (after 16 years); mean of six measured profiles, mean plus and mean minus standard deviation; best fit with equation (2) and prediction according to the DuraCrete model with equation (1)*

### 4.4 *Further analysis*

Most of the observed chloride surface 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 [Bamforth and Chapman-Andrews 1994] and for submerged exposure in the laboratory [Polder 1996]. In the present set of data, Haringvliet middle & high surface contents are much lower, which suggests that those two test areas do not belong to the splash zone group (see below). Interestingly, the surface content for Noordland changed from 1.8% after 8 years to 2.2% after 18 years. Apparently the surface content had not yet stabilised after 8 years. The value after 18 years is assumed to be the mature, steady value.

The effective diffusion coefficients in Table 4 can be compared to those obtained by profile fitting from specimens submerged in the North Sea at 5 or 100 m depth for 16 years [Polder & Larbi 1995]. Slightly depending on the age at first contact with seawater, OPC concrete with w/c 0.40 showed $D_t$ values of 1 to 3 * $10^{-12}$ $m^2/s$ and BFSC concrete with w/c 0.42 of about 0.3 * $10^{-12}$ $m^2/s$. It appears that the present diffusion coefficients for OPC concrete are much lower (0.1 to 0.3 * $10^{-12}$ $m^2/s$) and for BFSC similar to slightly lower (0.1 to 0.3 * $10^{-12}$ $m^2/s$) values than the results from submerged exposure. Neglecting minor differences of w/c and age, this suggests that Portland cement concrete in the splash zone has a much lower effective diffusion coefficient than in submerged exposure. An attempt is made to explain this.

*Table 4 Mean and standard deviations for surface chloride contents and effective diffusion coefficients from the best fitting curves; all structures 16 – 40 years old, except where noted; all results are based on six profiles, except Noordland (four for 8 years, three for 18 years age)*

| **Structure** | **Test area** | **Surface content (% chloride by mass of cement)** | | **Diffusion coefficient ($10^{-12}$ $m^2/s$)** | |
|---|---|---|---|---|---|
| | | μ | σ | μ | σ |
| Pier Scheveningen | Precast slab (+ 7 m) | 3.5 | 1.3 | 0.14 | 0.03 |
| Pier Scheveningen | Cross beam (+5 m) | 2.6 | 1.0 | 0.28 | 0.14 |
| Pier Scheveningen | In situ (+ 5 m) | 3.2 | 1.3 | 0.33 | 0.14 |
| Haringvliet | Pier 11, low (+ 1 m) | 2.8 | 2.0 | 0.12 | 0.04 |
| Haringvliet | Pier 11, middle (+9m) | 0.4 | 0.0 | 0.14 | 0.05 |
| Haringvliet | Pier 11, high (+14m) | 0.7 | 0.2 | 0.10 | 0.02 |
| Calandkanaal | Quay wall (+1 m) | 3.9 | 1.9 | 0.19 | 0.02 |
| Hartelhaven | Quay wall (+1 m) | 2.9 | 0.3 | 0.12 | 0.01 |
| Europahaven | Quay wall (+1 m) | 3.9 | 1.3 | 0.12 | 0.01 |
| SVKO | Pier Hammen 8 (+1m) | 2.2 | 0.6 | 0.24 | 0.07 |
| SVKO | Upper beam (+4m) | 4.1 | 0.3 | 0.27 | 0.06 |
| SVKO | Bridge element (+9m) | 5.3 | 1.5 | 0.28 | 0.12 |
| Noordland | Quay wall (+1 m), 8 years | 1.8 | 0.3 | 0.84 | 0.1 |
| Noordland | Quay wall (+1 m), 18 years | 2.2 | 0.1 | 0.36 | 0.0 |

In both the present fieldwork and the study on submerged specimens, the electrical resistivity of the concrete was measured using a four-point surface probe according to Wenner [Polder 2000]. Submerged OPC concrete specimens had resistivities from 100 to 200 Ωm [Polder & Larbi 1995], precast OPC field concrete at Scheveningen had resistivities measured on site of 300 to 500 Ωm. Concrete resistivity is very sensitive to moisture content [Polder 2000]. The increased resistivity suggests that OPC concrete in the splash zone in this investigation has dried out considerably, thus explaining the lower effective diffusion coefficients as compared to submerged OPC concrete. After 16 years submersion, BFSC concrete had resistivities of 400 to 1000 Ωm. The present results from the splash zone were either 300 to 600 Ωm (Pier Scheveningen, Haringvliet low, Eastern Scheldt Barrier) or between 1300 and 3300 Ωm

(Calandkanaal, Haringvliet middle and high); Hartelhaven resistivity was as high as 6300 Ωm. All quoted values are mean values, around which always considerable scatter was found (coefficients of variation were typically 10 to 30%). This suggests that the BFSC concrete in the former group of three test areas is rather wet, while the second group (three test areas) of concrete has dried out to some extent. Hartelhaven concrete was apparently quite dry when measured.

This tentative analysis suggests that drying out of Portland cement concrete, in the higher marine splash zone, may slow down chloride transport considerably as compared to transport in water saturated (submerged) 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 been investigated under equilibrium non-saturated conditions [Vera et al. 2004] and under simulated wetting and drying cycles with salt solution [Polder & Visser 2004]. Drying out, increased concrete resistivity and reduced transport rates were found to be related [Polder & Peelen 2002]. The effects of drying out of concrete on chloride transport, in particular in the splash zone, however, require more study.

## 5 Comparison to the DuraCrete model

The effective diffusion coefficients derived from the best fitting curves showed a time-dependency, qualitatively similar to that described in the literature [Maage et al. 1996]. Following the DuraCrete model, this time-dependency is described by:

$$D_t = K_{tot}\, D_0 \left(\frac{t_0}{t}\right)^{n_{Cl}} \qquad (3),$$

with $D_t$ the effective diffusion coefficient at the time of inspection ($m^2/s$), $K_{tot}$ a composite coefficient for the influence of cement type, environment and curing (-), $D_0$ the diffusion coefficient at reference time $t_0$ of 28 days ($m^2/s$), determined by Rapid Chloride Migration (RCM) testing and $n_{Cl}$ an aging exponent ($0< n_{Cl} <1$).

From the comparison of the field results to predictions from the DuraCrete model using equation (1), two deviations were proposed from the DuraCrete model and its input parameters. One deviation regards the environmental coefficient. The original calculation includes the cement type, the environment and the length of the curing period. With all cement being of one type (excepting Portland cement parts of Pier Scheveningen), the environment being rather homogeneous (marine tidal and splash zones) and the curing essentially unknown, a better way of calculating this coefficient was looked for. In the follow up on DuraCrete and using its database, Gehlen has proposed an environmental coefficient for concrete in marine environment based only on temperature [Gehlen 2000]. His main consideration is that in the tidal and splash zones, the influence of the length of the active curing period is relatively small because the concrete is kept wet due to its natural exposure. Acknowledging that proper curing after concrete casting should not be neglected, we have accepted Gehlen's formulation of the environmental coefficient, calculated by:

$$K_G = \exp\left[b_e\left(\frac{1}{T_{ref}} - \frac{1}{T_e}\right)\right] \quad (4),$$

with $b_e$ a regression parameter of about 4800 (K), $T_{ref}$ the reference temperature (293 K) and $T_e$ the annual mean air temperature (K). For a prevailing annual mean temperature of about 10°C, this parameter takes a mean value of 0.56 and a standard deviation 0.045 as given by [Gehlen 2000]. This simplification is attractive compared to the much more complicated formulation of the original DuraCrete environmental constant. It is emphasised, however, that this simplification is only allowed in marine conditions (for submerged, tidal and splash zones; and not to the atmospheric zone). The influence of the cement type is accounted for in another parameter, the ageing exponent $n_{Cl}$.

The second deviation for DuraCrete concerns the time dependency of the diffusion coefficient. Three groups of diffusion coefficients were analysed: data from our test areas (Table 4); data from additional chloride profiles for concrete in marine environment after 8 to 16 years including those from Noordland and specimens submerged in the North Sea [Polder & Larbi 1995]; and from RCM testing of laboratory specimens up to three years age. These latter were multiplied by $K_G$ as given above to transform them into effective diffusion coefficients. As presented in Figure 5, these three groups of data showed a satisfactory fit to a straight line in a plot of log D versus log t, with a (negative) slope of about 0.48 (standard deviation 0.07). For the combined datasets, an exponent of 0.48 best described the time dependency of the apparent chloride diffusion coefficient.

Consequently, a deviation was proposed regarding the value of the aging exponent. DuraCrete gives values for this exponent for Blast Furnace Slag cement concrete of 0.6 to 0.8. Most probably these relatively high values are based on data obtained from binders composed of Portland cement and slag that were added separately during concrete mixing [Bamforth & Chapman-Andrews 1994]. Such composite Portland-slag mixes may have different characteristics from Dutch slag cement, for which slag is intermixed with clinker during cement production and whose properties are carefully monitored. According to experience with Dutch Blast Furnace Slag cement, this exponent should be lower. As mentioned above, the best fit to our extended dataset was obtained using a mean aging exponent of 0.48. 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]. Taking into account the modified environmental coefficient and aging exponent, the (RCM) chloride diffusion coefficient at 28 days of the investigated concretes was inferred to be about 5.0 * $10^{-12}$ $m^2/s$. This value agrees well with laboratory test data for modern CEM III/B concrete with a w/c of about 0.5.

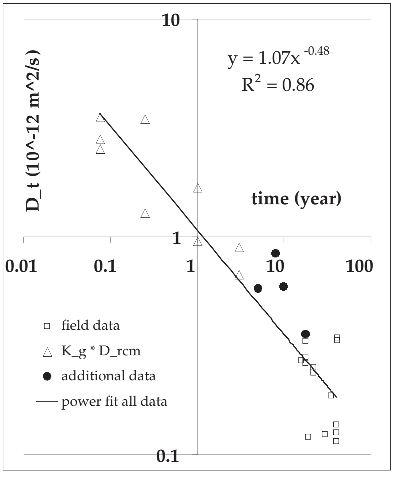


*Figure 5 Compilation of diffusion coefficients from field work in this study, additional exposure data and laboratory data with best fitting straight line; note log – log scale*

In our dataset the chloride surface content showed relatively little variation over the investigated structures, provided the height above sea level was no more than 7 meters and the age at which the profiles were taken was at least 10 years. The variation present in the surface contents can be incorporated by using a mean value of 2.9% chloride by mass of cement and a standard deviation of 0.8%. This is a deviation from the DuraCrete input, which is based on a surface content that depends on the water-to-cement ratio (w/c) and the exposure conditions. The variation of w/c in the structures investigated here is relatively low (approximately between 0.55 and 0.45), while the DuraCrete model intends to be more generally applicable (including higher and lower w/c's). The surface content presented here applies to the splash zone, in modern terminology environmental class XS3 according to EN 206-1.

For heights above + 7 m, two different patterns were present in the data. For the bridge elements of the Eastern Scheldt Barrier (at +9 m), high values of about 5% were found (Table 4).

This can be explained by considering that here the concrete is relatively sheltered from rain by the bridge deck above the test area (see Figure 3), so most of the chloride deposited on the concrete surface by splash and wave action is accumulated without being washed away. On the other hand, for the middle and high parts of the Haringvliet pier (+ 9 and +14 m), surface contents of 1% or lower were found. In these parts the concrete is fully exposed to rain and at a relatively high distance from seawater splash and waves, so the balance between accumulation and washout is the other way around than in the Eastern Scheldt Barrier.

The model with the proposed modifications is termed the DuMaCon version of the DuraCrete model. It must be stressed that this model and its modified input parameters apply only to concrete made with Dutch slag cement with at least 65% of slag, in modern terminology CEM III/B LH HS, exposed to marine environment (exposure classes XS3, XS2).

Another issue to be considered is the critical chloride content. This is the value above which it is assumed that the reinforcement will start to corrode. Many values have been published in the literature. Nowadays it is well accepted that there is no single value; rather a range of values describes an increasing probability of corrosion (initiation). Practical observations have shown that about 0.5% chloride by mass of cement is a good estimate of the value at which the probability of corrosion is about 50%. However, these observations concern Portland cement concrete. There is insufficient practical evidence for specifying a reliable value for the critical chloride content for Blast furnace Slag cement concrete in practice. Consequently, in our study we have used the value for OPC concrete ($\mu$ 0.5%, $\sigma$ 0.15%).

# 6 Conclusions: application of the DuMaCon model

From extensive fieldwork and additional data on Dutch concrete in marine exposure, a modification of the DuraCrete model for chloride transport in slag cement concrete in marine environment was derived. Modifications include a fixed chloride surface content, a new value for the ageing exponent and a different way of calculating the environmental coefficient. Statistical parameters have been specified.

This DuMaCon version of the DuraCrete model can be used for service life assessment of concrete structures in marine environment made of CEM III/B, *in several ways*. Firstly it can be used as a *service life design model* for new structures. The chloride diffusion coefficient can be tested (using the Rapid Chloride Migration test) on a trial mix and the surface content, the aging exponent and the environmental coefficient can be taken from the DuMaCon model. With that input, the chloride ingress can be predicted and the cover to the reinforcement can be chosen such that corrosion initiation is postponed until (at least) the end of the desired service life. Alternatively, the cover depth is chosen and a maximum value for the diffusion coefficient is calculated. Given that the standard deviations and distribution types of all parameters are known (either from DuraCrete or from the present study), full probabilistic analysis is possible, allowing to calculate failure probabilities (see below).

Secondly, the DuMaCon version can be used as a *service life REdesign* model for assessment of existing structures. Two options exist: the "desk option" and the "inspection option". For the *desk option*, one only needs to estimate the diffusion coefficient at young age (for example from a database) and to predict chloride ingress as for a new structure using the DuMaCon input parameters. The cover depth can be taken from design documents, assuming a standard deviation of about 10 mm [DuraCrete R17, 2000]. However, it is better to measure the actual cover depth but that cannot be done from behind a desk. From the cover depth the point in time when initiation most likely will occur can be calculated. Knowing the structure's present age, one can estimate how far it is from corrosion initiation. For the *inspection option*, one needs to inspect the most critical areas of the structure, measure cover depths, take cores and perform chloride profile analysis. Chloride surface contents and effective diffusion coefficients are found by fitting equation (2) to the profiles. The surface content can be regarded a constant and the effective diffusion coefficient will decrease by the aging exponent given above. Again, the point in time when corrosion initiation occurs can be calculated. The *inspection option* for REdesign requires more work (and costs) than the *desk option*, however, it is more accurate because it is based on the real cover depth and the response of the structure to its actual environment. Again, using mean values, standard deviations and distribution types of all parameters, probabilistic calculations can be made to produce failure probabilities or reliability indices.

# 7 Recommendations and outlook

As described above, chloride ingress into concrete in marine environment was found to be subject to considerable scatter, both on micro and macro scales of observation. In practical terms, this suggests that relatively large numbers of samples should be taken to get reliable results. Six cores in a small area and analysed at six depths, as applied in this study, probably is a good minimum number to start with. In more general terms, all variations should be taken into account and full probabilistic analysis is required. Not until a lot more experience has been obtained, partial safety factors can be derived and semi-probabilistic analyses can be made with confidence.

A further issue that warrants attention is the limit state and the failure probability that should be associated with it. In the approach presented, corrosion initiation is the undesired event whose occurrence is predicted (and which should be delayed for a prescribed period). However, corrosion initiation it is not the end of the service life. It is a serviceability limit state, not an ultimate limit state. Depending on the type of structure, some level of corrosion can be tolerated. Knowing when in the future corrosion initiation will occur provides the opportunity to start thinking about maintenance well in time. If chloride has not yet penetrated very close to the reinforcement, preventative maintenance may be the best option, e.g. applying coatings or hydrophobic treatment [Polder et al. 2001]. Even if corrosion is expected relatively soon, it is useful to have some years to prepare for selecting protection options, e.g. conventional repair or cathodic protection [Polder 1998].

It is clear that the failure probability associated with the limit state “corrosion initiation” should be less than 50%. Considering the scatter in chloride ingress (implying the presence of weak spots), at the point in time when there is 50% chance of reinforcement corroding, a part of the reinforcement will already corrode and significant damage may already have occurred. As repair is costly, economic considerations should be made of the acceptable failure probability. In the examples mentioned of service life design for large tunnels, a failure probability was required of about 3% (reliability index $\beta=1.8$) [Siemes et al. 1998]. For more ordinary structures, this may be too strict. A level of 10% ($\beta=1.3$) was proposed in Norway [Fluge 2001]. In any case, the event of corrosion initiation should be considered a *maintenance limit state* and further work is required to elaborate its associated failure probability.

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