Guideline for service life design of structural concrete - A performance based approach with regard to chloride induced corrosion

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Nowadays major structures require a service life of 100 years or even more. CUR committee VC81 has developed a guideline for service life design based on initiation of corrosion due to chloride penetration for exposure classes XD or XS. A semi-probabilistic simplification of the DuraCrete methodology is introduced with a probability of failure of ≤ 10% for corrosion initiation.

Three options are provided to the designer: (1) a range of concrete cover depths adapted to the binder type and exposure class; (2) a semi-probabilistic approach using a safety margin for the concrete cover depth; (3) full probabilistic calculations based on specified input parameters. The guideline permits optimisation with respect to cover depth and concrete composition.

The required concrete performance is the chloride diffusion resistance, measured as Rapid Chloride Migration. Its limit value depends on cover depth, required service life, exposure class and binder type (Portland, Blast furnace slag and Portland / fly ash cement). Model calculations were validated using data from marine and road structures.

Keywords: Concrete, reinforcement corrosion, chloride, service life design, performance

1 Introduction

Today many owners require service lives of 80, 100 or even 200 years for important concrete structures. Most present design codes do not give quantified guidance for designing for such long service lives. Usually a service life of 50 years is only implicitly assumed. Opportunity to improve this was offered by developments in performance based service life modelling in the 1990s. This led to installing an industry-wide committee in the Netherlands in 2003, whose task it was to develop a performance and probability based guideline for designing durable civil engineering structures with service lives up to 200 year. Due to limited experience it was agreed that the requirements of the prevailing Dutch concrete standards should be met, which correspond to international regulations (EN 1992-1-1, EN 206, EN 13670). This implies the usual maximum water-to-cement ratios and minimum cement contents, depending on environmental class. Under these conditions chloride-induced rebar corrosion is likely to be the dominant mechanism determining the service life, whereas carbonation-induced corrosion can be ruled out safely.
The guideline is based on a probability oriented methodology for Service Life Design (SLD) that was conceived in the 1980s by Siemes et al. (1985) and developed in detail in the 1990s in European research project DuraCrete (DuraCrete 2000, Siemes et al. 2000). It follows structural limit state design philosophy by stating that the service life is the period in which the structure's *resistance* R(t) can withstand the environmental *load* S(t). Other than for the usual structural design, both are (or can be) time dependent, statistically distributed variables. A particular *performance* is predicted with a predetermined maximum probability of failure at the end of the design life, as shown schematically in Fig. 1. DuraCrete proposed models for carbonation and chloride induced corrosion, and for cracking due to corrosion. The present Guideline is limited to chloride induced corrosion only. The limit state is initiation of reinforcement corrosion due to ingress of chloride ions. When the chloride content at the surface of the reinforcing bars exceeds the critical chloride threshold, the structure is considered to fail. Consequently, the occurrence of corrosion initiation is given by the balance between the actual chloride content at the steel surface and the critical chloride content. In the time dependent limit state function, the aggressive *load* term is represented by the chloride content at the steel surface that increases with time due to chloride ions building up at the concrete surface and their subsequent transport into the bulk. The *resistance* term is the critical corrosion initiating threshold for chloride at the steel surface. This latter property depends on many parameters, like type of cement, water/cement ratio, moisture content, etc. Literature data concerned are not consistent and

therefore cause significant uncertainty; it seems that it cannot reliably be influenced with present day knowledge (Angst et al. 2009, Polder 2009). No widely accepted test method is available, which means that the performance of a particular concrete or structure cannot be determined by testing. Hopefully work in RILEM Technical Committee 235-CTC 'Corrosion initiating chloride threshold concentrations in concrete' will improve this issue. As a practical approach, the critical chloride content is assumed constant in time and independent of binder type; its statistical distribution is based on literature data (Breit 1998, 2005) that appear to correspond reasonably well with field data (Vassie 1984), see also (Gehlen 2000, Gaal et al. 2003).

This leaves determining the performance of individual concretes or structures to chloride transport through the concrete cover. Chloride transport is the result of a driving force, determined by the environment that produces a particular chloride content on the concrete surface; and the concrete resistance to chloride penetration. The latter is expressed by its chloride diffusion coefficient. This diffusion coefficient changes with time due to hydration, binding of chloride and drying out, which complicates the modelling.

It is emphasized that the *performance* is considered in terms of absence of corrosion initiation, which is not an Ultimate Limit State (ULS), because no direct danger for human lives or extreme economic losses is at stake. It is a Maintenance Limit State (MLS), because corrosion means the upcoming need to repair, which is an economic threat rather than a safety issue, corresponding to a Serviceability Limit State (SLS). Target probabilities of failure for SLS are usually in the range of a few to 10 percent.

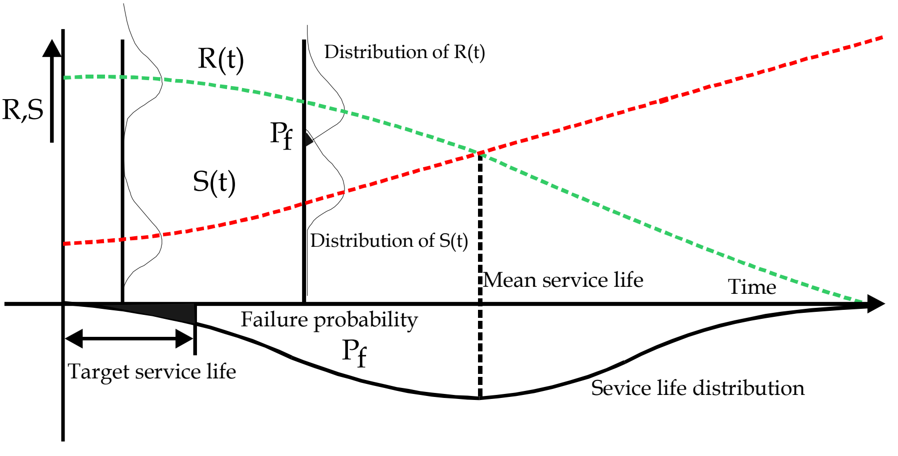

Figure 1: Schematic representation of probabilistic service life design (DuraCrete 2000, Siemes et al. 2000)

The DuraCrete model has been tested in the field on six marine structures on the Dutch coast, from which chloride profiles were taken after 20 to 40 years (Polder & Rooij 2005, Rooij & Polder 2005), from which modifications of model parameters were deduced. This paper describes application of the DuraCrete approach for service life design of concrete structures in marine (exposure classes XS) and de-icing salts environments (XD). Chloride migration test results are taken as the required performance of a particular concrete composition, which is used in a semi-probabilistic concept. The performance requirements were condensed into a set of tables specifying maximum values for chloride migration coefficients depending on cover depth, cement type, exposure class and required service life.

2 A probability-based predictive model

As mentioned above, the basic components of the Guideline's concept are a transport model, a chloride transport coefficient, several model parameters and a semi-probabilistic approach. For the description of chloride ion transport into concrete a modification of Fick's 2nd law of diffusion is adopted. Using Fick's 2nd law and the term diffusion does not imply, however, that the authors are not aware of other mechanisms that contribute to the transport of chloride ions into concrete (e.g. capillary absorption at less than full saturation), nor of more advanced models for chloride transport. That discussion, however, is not the subject of this paper. The effect of surface moisture fluctuations and degree of saturation of the bulk concrete is taken into account in the values for the surface chloride content and ageing coefficient, respectively.

2.1 Chloride diffusion coefficient obtained from RCM test

Since diffusion modelling for chloride ingress in concrete was introduced by Collepardi (1972), several methods have been proposed for determining the resistance of concrete against chloride penetration. They differ by the nature of the tested material (hardened cement paste, mortar of concrete), the sample thickness and consequently, the test duration, which is inevitably longer for necessarily thicker samples. For a practical guideline, testing concrete is considered imperative. In the 1990s, two methods for testing concrete were standardised in the Nordic countries:

- An immersion (pure diffusion) test, NT Build 443;
- An accelerated (migration) test, NT Build 492, the Rapid Chloride Migration test (RCM)

The immersion test may be seen as a realistic representation of natural diffusion. A drawback is that it requires seven weeks exposure of specimens and involves chloride analysis of many samples. The accelerated test involves a different transport mechanism, migration of ions driven by an electric field, but it has a short execution time and is less laborious. In the European research project CHLORTEST (CHLORTEST 2005, Tang et al. 2012) both methods were compared in a Round Robin Test. A good linear correlation was found between chloride diffusion coefficients from diffusion experiments and chloride migration coefficients from RCM tests. Thus, it was considered justified to use the less time consuming RCM test instead of the diffusion test (Breugel et al. 2008).

In the past few years, RCM testing has been applied in the Netherlands to many concrete mixtures in association with service life design of large infrastructural projects. For the guideline, a total of 500 RCM-values from 153 different concrete compositions (precast and ready mix) were obtained. The influence of the mix composition on the RCM-value was analysed. Cement types used were mainly Portland cement (CEM I 32.5R, 52.5N, 52.5R), blast furnace slag cement (CEM III/A 52.5 R, CEM III/B 42.5 LH HS), mixtures of these two cements and binders containing powder coal fly ash and Portland or slag cement. Binder contents ranged from 300 to 450 kg/m^3 and water/binder ratios (w/b) from 0.33 to 0.65. The age at testing ranged from 28 days to 3 years, with most data at an age around 28 days.

Starting point of the analysis was that RCM-values depend mainly on the type of binder (cement type and reactive additions), w/b and age. The data were first grouped with respect to binder type: Portland cement (CEM I); Slag cement (CEM III/A or III/B, 50 – 76% slag); Portland and slag mixtures (25 – 38% slag); Portland cement with fly ash (21 - 30% fly ash).

Within these binder type groups, data of similar age were aggregated; ages of 28 to 35 days were considered as a single group. The influence of w/b on DRCM was then analysed for each binder group at an age around 28 days. It should be noted that all fly ash present was considered as cementitious material, so w/b = water / (cement + fly ash). The analysis showed that the D_{RCM} (28d)-value is linearly related to the water/binder ratio:

$$D_{\mathrm{RCM}}\,(28\mathrm{d}) = A(\mathrm{w/b}) + B, \tag{1}$$

with A and B constants for particular cement types. The results displayed in Figure 2 clearly show that both the level of D_{RCM} and its w/b dependency strongly depend on binder type. For Portland cement, D_{RCM} is strongly influenced by w/b. For slag cement this influence turned out to be less pronounced. The regression coefficients (A, B) found for

different binders are in good agreement with those reported by Gehlen (2000). The data suggest that in the range of practical w/b ratios between 0.35 and 0.55 (and present day concrete technology), minimum RCM-values apply that depend on binder type.
As any one method, the RCM test produces variations in its test results, typically of 20% (Tang & Sørensen 2001). For a given concrete composition the scatter increases with the maximum grain size of the concrete (D_{max}). Consequently, a minimum number of six samples of 100 mm diameter is required for D_{max} 32 mm and of three for D_{max} 16 mm.

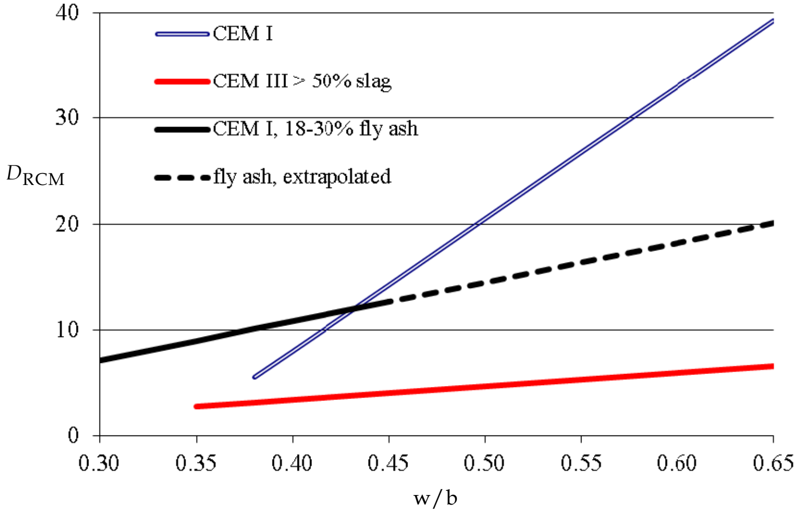

*Figure 2: Correlation between w/b (-) and D_{RCM} at about 28 days; D_{RCM} values * 10^{-12} m^2/s*
Note: this figure is printed in greyscale; colours are available in the online version.

2.2 ***Correlated method***

Even though execution of the RCM-test is much faster than the diffusion test, it is still time and labour intensive. For regular production control, there is a need for a good but quick indicator of the resistance against chloride ingress. As suggested by DuraCrete (2000) the Two Electrode Method (TEM) for concrete resistivity testing can be used. This is a quick and simple non-destructive test that can be applied on any regularly shaped specimen (Polder 2000). Correlation between chloride transport in and resistivity of concrete was theoretically underpinned and practically demonstrated in the 1990s (Andrade et al. 1993, 1994, Polder 1997). Thus, the TEM test is applied for production control to standard concrete cubes for compressive strength testing after wet curing at an age of 28 days. Positive experience was gained using this method for quality control of cast in situ concrete for parts of the Green Heart Tunnel in the high Speed railway line between Amsterdam and Brussels (Rooij et al. 2007).

2.3 *Modelling chloride ingress*

In the DuraCrete model the time-development of chloride profiles is approximated by:

$$C(x,t) = C_s - (C_s - C_i)\ erf \frac{x}{\sqrt{4\,k\,D(t)\,t}}\ , \tag{2}$$

with *C(x,t)* the chloride content (all chloride contents are expressed in % by mass of binder) at depth *x* at time *t*, C_s the surface chloride content, C_i the initial chloride content in the concrete, *k* a correction factor and *D*(*t*) the apparent diffusion coefficient, which is a function of time. The surface chloride content was assumed to be independent of mix composition for reasons of simplicity. It does depend on the environment: based on extensive studies of Dutch marine structures a value of 3.0% was used for XS environments (Polder & Rooij 2005, Rooij & Polder 2005) and from a study on bridges (Gaal 2004) a value of 1.5% was adopted for Dutch land based structures in XD environments. The initial chloride content was taken equal to 0.1% based on data from concrete suppliers.

The apparent diffusion coefficient *D*(*t*) is multiplied by a correction factor *k* to obtain the chloride diffusivity of concrete in a real structure. This correction factor depends on binder type, environment and duration of wet curing. The *k*-values were taken from DuraCrete (2000). The critical chloride content was taken equal to 0.6% by mass of cement based on (Breit 1998, 2005). As mentioned before there is no general consensus about the specific value(s) for the critical chloride content and hence the latter value was used as a best estimate. If desired a different value can be applied in the model of the guideline.

2.4 *Time dependency of chloride penetration coefficient D(t)*

The rate at which chloride ions penetrate into concrete decreases with time. This is due to progressing hydration of the binder, which causes narrowing of capillary pores (especially in binders with slag or fly ash), and drying, which reduces the amount of liquid in the pores. A decreasing rate of chloride penetration can be described with a time-dependent diffusion coefficient (Maage et al. 1996):

$$D(t) = D_0 \left(\frac{t_0}{t}\right)^n\ , \tag{3}$$

with D_0 the D_{RCM}-value at reference time t_0 (usually 28 days) and n the ageing coefficient ($0 < n < 1$). The value of n depends on the rate of hydration and on the extent of drying and thus on the type of binder and type of environment. Ageing coefficients for different binders without the effect of drying were determined from DRCM-values that had been cured under water at 20°C for different periods from 28 days up to three years. Structures in the field will dry out to some extent and hydration may occur slower than under water. In DuraCrete ageing coefficient values under field conditions were determined from profiles taken from structures and exposure tests, combined with k values (Equation 2) for different environments and duration of wet curing. Based on those data, our own analysis and additional work (Polder & Rooij 2005, Rooij & Polder 2005), n-values were chosen for the Guideline in two groups of environmental classes: very wet (XD2/XS2/XS3) and moderately wet (XD1/XD3/XS1) (CUR 2009), see Table 1.

Table 1: Ageing coefficients n for different binders in two groups of environmental classes

Environmental classes	Underground, splash zone XD2, XS2, XS3	Above ground, marine atmospheric XD1, XD3, XS1
Type of binder		
CEM I	0.40	0.60
CEM I, 25-50% slag, II/B-S; III/A, <50% slag	0.45	0.65
CEM III/A or /B, 50-80% slag	0.50	0.70
CEM I with 21-30 % fly ash	0.70	0.80
CEM V/A with c. 25% slag and 25% fly ash	0.60	0.70

2.5 *Reliability considerations and semi-probabilistic approach*

For a given concrete cover depth, Equation (2) can be used for calculating the time needed for the critical chloride content to reach the reinforcement. Such a calculation, however, is deterministic and yields a mean value. This means that the probability of corrosion initiation at that point in time and space is 50%. In practice such a high probability is unacceptable, as it would mean that weak spots suffer corrosion much earlier and interventions may already be needed well before the intended end of the service life. An acceptable probability of failure for corrosion initiation of reinforcing steel may be 10% (Fluge 2001).

To obtain such a lower probability of failure than 50%, either the cover depth can be increased or the maximum D_0 can be decreased. If the former option is chosen, the required amount of additional cover can be calculated for each individual case using probabilistic calculations. In the guideline, however, it was chosen to add a fixed amount to the (deterministically determined) minimum cover depth *as a safety margin*. This is a semi-probabilistic approach, comparable to using a safety factor for a materials property or a load.

An increase of the cover depth by 20 mm will reduce the probability of corrosion initiation from 50% to about 10% for the typical range of cover depths for bridges and marine structures (Breugel et al. 2008). This procedure has been followed in the guideline. Obviously, at a lower initial cover depth an increase of the concrete cover with 20 mm will result in a lower probability of corrosion initiation than for a higher initial cover depth. For cover depths in the range of 35 to 60 mm, including a safety margin of 20 mm, calculations were performed using TNO's probabilistic software Prob2B™ on a set of example cases with different binder types and w/b ratios. The resulting probabilities of failure ranged from 6% to 16% with an average of 10% (CUR 2009). A safety margin of 30 mm produces a probability of 4% on average (with a range of 1 - 7 %). Such probabilities are considered appropriate for reinforcing and prestressing steel, respectively. The additional cover of 10 mm on prestressing steel relative to ordinary reinforcement is more than prescribed in the European standard EN 1992-1-1, where it is only 5 mm.

3 Service life design in practice - examples

Following the method described above, including a safety margin to the cover depth of 20 mm for reinforcing steel or 30 mm for prestressing steel, combinations of required cover depth and maximum DRCM-values were calculated for service lives of 80, 100 or 200 years. Curing was assumed to last for three days (XD) or seven days (XS). Values for 100 years are presented in the Table 2. Two examples may illustrate using it for service life design.

Example I concerns a reinforced concrete structure in XD1-3/XS1 environment. For the type of cement CEM III/B with 70% slag was chosen. The required service life is 100 years. In Table 2 it can be seen that with a cover depth of 45 mm (to the reinforcing steel), a maximum $D_{RCM,28}$ is required of $6.0 \cdot 10^{-12}$ m²/s. With this cement and a w/b of 0.45, a D_{RCM}-value of $4.0 \cdot 10^{-12}$ m²/s can be obtained rather easily (Figure 2). Going back to Table 2 it can be seen that with a D_{RCM}-value of $4.0 \cdot 10^{-12}$ m²/s the cover depth could be reduced to 40 mm.

Example II concerns the same structure as Example I. The cover depth is 45 mm, but now CEM I is used. For CEM I and a cover depth of 45 mm, Table 2 gives a maximum $D_{RCM,28}$ of $8.5 \cdot 10^{-12}$ m²/s. Even though D_{RCM} is higher than in case I, such a value might be hard to achieve with CEM I (see Figure 2). It would require quite a low w/b, probably below 0.4, which may cause workability problems. Increasing the cover to 50 mm will allow an increase of $D_{RCM,28}$ to $12 \cdot 10^{-12}$ m²/s, which can be readily achieved with a w/b of about 0.45. Navigating through all possible options in the tables, the designer can find the economic optimum, while he can demonstrate to the client that the required service life is achieved.

4 Feedback from practice

In order to evaluate and if necessary to improve this guideline feedback from practical applications was requested. The (limited) feedback obtained up to now confirms the applicability of this guideline as is demonstrated by the following example. For a tunnel with a required service life design of 100 years in XD3 environment concrete with cement type CEM lll/B was chosen. This concrete exhibits an initial chloride content of 0.11% m/m by mass of cement and a $D_{RCM,28}$-value of $5.8 \cdot 10^{-12}$ m²/s. Based on this guideline the

Table 2: Maximum $D_{RCM,28}$ for various cover depths as a function of binder type and environmental class for a design service life 100 years.
Note: Boldface values are practically achievable by present-day concrete technology with currently used w/b. Italic values are not achievable (lower values) or not recommended (higher values)

Mean cover [mm]		Maximum value $D_{RCM,28}$ [10^{-12} m²/s]							
		CEM I		CEM I+III 25-50% S		CEM III 50-80% S		CEM II/B-V, CEM I+20-30% V	
Reinforcing steel	Prestressing steel	XD1 XD2 XD3 XS1	XS2 XS3	XD1 XD2 XD3 XS1	XS2 XS3	XD1 XD2 XD3 XS1	XS2 XS3	XD1 XD2 XD3 XS1	XS2 XS3
35	45	*3.0*	*1.5*	*2.0*	*1.0*	**2.0**	*1.0*	*6.5*	*5.5*
40	50	**5.5**	*2.0*	**4.0**	*1.5*	**4.0**	*1.5*	**12**	**10**
45	55	**8.5**	*3.5*	**6.0**	*2.5*	**6.0**	**2.5**	**18**	**15**
50	60	**12**	**5.0**	**9.0**	*3.5*	**8.5**	**3.5**	**26**	**22**
55	65	**17**	**7.0**	*12*	**5.0**	*12*	**5.0**	*36*	*30*
60	70	**22**	**9.0**	*16*	**6.5**	*15*	**6.5**	*47*	*39*

minimum concrete cover has been calculated as a function of the curing period (Table 3). The prescribed minimum concrete cover of 50 mm for this concrete structure is higher than all calculated values and hence is adequate. During the execution of the project the origin of the CEM lll/B cement was changed, resulting in a higher $D_{RCM,28}$-value of $6.8 \cdot 10^{-12}$ m²/s. This corresponds to an increase in calculated minimum concrete cover of 3 mm, which (if properly cured) is still below the prescribed minimum concrete cover of 50 mm (Table 3).

Table 3: Calculated minimum concrete cover as function of curing period for particular case (see text)

Curing period (days)	Curing factor (DuraCrete) (-)	Minimum cover (mm)
1	2.08	49
3	1.5	45
7	1.0	40

Although the prescribed curing was at least 4 days in the moulds or applying an adequate curing compound immediately after demoulding, a small part of the structure was demoulded after 1 day without being treated with a curing compound. D_{RCM}-values measured on concrete cores taken from this part of the structure were a factor 2.2 higher than those measured on well cured concrete cubes of the same composition and age. This factor 2.2 is very similar to the ratio of DuraCrete curing factor between 1 and 7 days (= 2.08).
The variation coefficient of D_{RCM}-values measured on cores taken from the structure is similar to those measured on laboratory cured concrete cubes of the same composition, which is 15 to 20%.

5 Concluding remarks

Obtaining the required service life in aggressive (chloride contaminated) environments for structural concrete (bridges, harbour quays, tunnels, parking garages) is a matter of postponing the onset of reinforcement corrosion for a specified period of time. The most important elements of the proposed model based performance approach are:

- corrosion initiation due to chloride ingress is the design limit state; hence, the corrosion propagation phase is not taken into account (conservative approach)

- chloride load from sea water or de-icing salt environment drives ingress
- chloride is transported in concrete by diffusion; other transport mechanisms are partly taken into account by the effective value for the surface chloride content and the ageing coefficient
- the diffusion coefficient is time dependent
- environmental, curing and materials influences are taking into account by specific coefficients
- the transport coefficient of the intended concrete is determined by Rapid Chloride Migration testing.

This paper presents a probability and performance based design procedure for determining combinations of cover depth and 28-day chloride migration coefficients that are required to achieve a specified service life. Target probabilities of failure are 10% and 5%, respectively, for reinforcing steel and prestressing steel. Based on a semi-probabilistic simplification, the required combinations of cement type, cover depth and migration coefficient are brought together in simple design tables. The tables give limiting values for chloride diffusion coefficients obtained with the RCM test for service lives of 80 to 200 years in marine (XS) or de-icing salt (XD) environments. From analysis of a large number of test results, the dependency of the DRCM-value on w/b and cement type was determined and an indication was obtained which values are possible using present-day concrete technology.

Similar tables as proposed here have been presented by (Li et al. 2008). In their tables, however, the compressive strength is still considered one of the durability parameters. Instead of the strength, here an explicit transport parameter is chosen, i.e. the RCM-value, to indicate the concrete's performance in terms of susceptibility to chloride ingress. A similar probability-based approach to various degradation mechanisms has been presented by *fib* (2006), using a slightly different model for chloride induced corrosion.

With this Guideline (CUR 2009), the Dutch concrete industry now has rules for practical service life design. All parties involved have agreed to collect their experience using the Guideline, with the intention to evaluate it and if necessary, to improve it in the near future. At the same time, however, it was realised that many items used in the calculations still contain large uncertainties. Further research should contribute to reducing them.

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