

# Procedure for determining the remaining time to initiation of chloride induced reinforcement corrosion for existing concrete structures

Rob B. Polder<sup>1,2,3,\*</sup>, Michel Boutz<sup>4</sup>, and Marc Ottelé<sup>2,5</sup>

<sup>1</sup>RPCP, Gouda, The Netherlands, <sup>2</sup>Delft University of Technology, Delft, The Netherlands, <sup>3</sup>Leeds University, Leeds, UK  
<sup>4</sup>SGS INTRON, Sittard, The Netherlands, <sup>5</sup>Heijmans Infra, Rosmalen, The Netherlands

**Abstract.** For efficient asset management of reinforced concrete infrastructure, owners want to know when to expect corrosion in structures. At present, no widely accepted procedures for testing of existing structures for the remaining time to corrosion initiation are available. This paper describes such a procedure, based on the authors' long-time experience. From about 20 years age, the concrete contains its mature response to actual environmental loads, e.g. chloride ingress. By measuring the mean and standard deviation of the actual cover depth, taking chloride profiles, assuming a few parameters and a simple, pragmatic model, the expected time to corrosion initiation for a particular test area can be predicted. Required numbers of cores and samples per core are given. Uncertainties are taken into account by subtracting a safety margin from the measured concrete cover depth. Results of six cores per test area are classified and interpretations are given. Because of large variability, the results are classified in three ranges of time to corrosion initiation: five years or less, five to fifteen years, or more than 15 years. The procedure has been approved by the relevant national Standards committees and was issued as a CUR Recommendation in 2018. It is illustrated based on a field case and the obtained results are discussed.

## 1 Introduction

Concrete infrastructure in many parts of the world is exposed to chlorides derived from de-icing salts or sea water, which may cause reinforcement corrosion at some point in their life [1]. Chloride induced corrosion is the main degradation mechanism limiting the service life of these structures. The majority of bridges in The Netherlands has been built between 1960 and 1980. Various studies have suggested that chloride induced reinforcement corrosion appears in increasing numbers of motorway bridges from ages of about 40 years on [2, 3], potentially increasing the number of cases that need repair and/or protection. Anticipating which structures will develop corrosion at which point in time is an increasingly important need for asset managers. Nowadays models, required input parameters and test methods are available for designing new structures for long service lives [4, 5, 6]. These models aim at a pre-set maximum probability of corrosion initiation at the end of the desired service period. However, widely accepted and published approaches to test existing structures for the remaining time to corrosion initiation are lacking. This paper reports such an approach, based on a CUR Technical Recommendation set up by SBRCURnet committee 2140 "Remaining service life of existing concrete structures" [7]. The procedure is based on existing practice of a number of persons and organisations with condition-oriented inspection, on site

testing, sampling and interpretation for existing concrete structures, among others [8, 9]. The procedure allows for taking either carbonation or chloride induced corrosion into account. The Recommendation also addresses some aspects of structures with corrosion damage, i.e. in the propagation period. However, this paper deals with initiation of chloride induced corrosion only, since it is the most practical limit for the service life in most cases. It is emphasised that the Recommendation is aimed at application in practice, and does not include the latest scientific views and findings, but rather is focused on obtaining results supporting asset management in the field.

## 2 Basic considerations

Basic considerations are as follows. The procedure is intended for application to structures in marine (exposure classes XS1, XS3) and de-icing salt (XD1, XD3) environments. In The Netherlands, this applies to all concrete structures in coastal and road environment. Existing structures have a time-independent (and stochastic) distribution of cover depth. If they are exposed to de-icing salts (or sea salt), it will take at least ten to twenty years before they will contain a fully developed set of time-dependent parameters, such as chloride penetration resistance, response to the actual environmental load (chloride surface content and

\* Corresponding author: [ropolder@ropolder.demon.nl](mailto:ropolder@ropolder.demon.nl)

penetration), moisture distribution and carbonation, all with their particular spatial distributions. For simplicity they are assumed to be constant in time, except chloride penetration. Cover depths and chloride penetration profiles can be measured relatively easily. However, sample size and numbers need to be considered due to significant spatial variation resulting from execution, environmental loads and those inherent to the presence of aggregate [8, 10]. Values for the critical chloride content can be based on experience or can be tested following a simplified pragmatic approach. A simplified model for chloride transport based on a constant surface content and a constant diffusion coefficient is provided, because such a model seems to be most effective with regard to concrete properties and environment/concrete interaction as reflected in the measured chloride profile. The procedure is aimed at corrosion initiation, the end of the initiation phase, as a proxy for “end of service life”, which obviously neglects the propagation phase of reinforcement corrosion. At present it is considered impossible to predict the propagation of corrosion and its consequences for structural behaviour with sufficient accuracy for application in practice. Consequently, this procedure assumes that corrosion has not yet been initiated and the structure does not have concrete damage due to corrosion.

### 3 Steps to be followed for application of the procedure

The following steps have to be taken when a structure is investigated according to the present procedure:

- Analysis of existing information such as year of construction (the structure should be at least 10 to 20 years old), design and as-built drawings, previous inspections and repair measures, verification of exposure environment;
- Identification of critical parts, in particular load bearing elements and areas with increased corrosion risk, based on historic information and previous inspection reports;
- Visual inspection for damage, such as cracking, spalling, signs of corrosion and other relevant phenomena, e.g. leakage of joints. If inspection results indicate that damage occurs due to other mechanisms than corrosion (e.g. ASR), the remaining service life cannot be determined by the procedure described here;
- Selection of at least six Test Areas (TA) within the critical parts, of typically 1 x 1 m<sup>2</sup> or 2 x 0.5 m<sup>2</sup>.

In each Test Area:

- A visual inspection is carried out for verification of previous inspections and selection of testing and sampling locations;
- Concrete cover depth ( $x$ ) is tested of at least twelve outer bars surrounding each coring location, from which a mean value  $x_m$  and a standard deviation is calculated

- (Optional) potential mapping to select coring locations for “worst spots”;

- Carbonation depths are determined using phenolphthalein on a minimum of six locations: preferably on cores (in the lab, following EN 14630) or in situ on freshly broken chips or drilled holes;

- Cores are taken for chloride profiles: a minimum of six cores, which is considered a reasonable compromise between effort and statistical representativeness based on previous work [8], of minimum diameter 50 mm (preferred 70 mm) and minimum depth of 60 mm (preferred 80 mm).

Then, in the laboratory:

- Cores are cut in at least six slices of 10 mm thickness (or a lower thickness if low ingress is expected)

- Each slice is crushed and milled to a particle size below 250 µm and dissolved in acid, the residue is weighed and chloride is determined in the filtrate; the chloride content is calculated by mass of binder from each sample’s acid soluble versus insoluble fractions including 20% of hydration water following EN 14629 [11].

- (Optional) cement type is determined, visually or using thin section microscopy.

The calculations in order to obtain the time to corrosion initiation are described in the next chapter.

### 4 Calculations

For each core, a chloride penetration profile is obtained to which a diffusion profile [12] is fitted using the least squares method to:

$$C(x, t) = C_s - (C_s - C_i) \operatorname{erf} \left[ \frac{x}{\sqrt{4 D_a t_{\text{insp}}}} \right] \quad (1)$$

where  $C(x, t)$  is the chloride content (all chloride contents in % by mass of binder) at depth  $x$  (m) and at the age of the structure when inspected  $t_{\text{insp}}$  (s);  $C_s$  is the chloride surface content (%);  $C_i$  is the initial chloride content (%);  $\operatorname{erf}$  is the error function;  $D_a$  is the apparent chloride diffusion coefficient (m<sup>2</sup>/s). The input is  $C(x, t)$  for at least six combinations of  $C$  and  $x$ , from each core, for  $t_{\text{insp}}$ ; and the output of the fit is  $C_s$ ,  $C_i$  and  $D_a$  for each core (profile).

Some profiles may have a lower chloride content in the first slice than in the next slice; in such cases the first point is neglected. In rare cases more than one point can be below the maximum and need to be neglected, which can negatively affect the accuracy of the results.

The fitting parameters  $C_s$ ,  $C_i$  and  $D_a$  are taken as constants, which may be conservative, in particular for  $D_a$  which may have a decreasing tendency over time, as has been found for slag cements.

Reflection on the results of the fitting may be useful and some reference is provided.  $C_s$  generally tends to lie in the range of 1-3%,  $C_i$  in concrete made using non-contaminated raw materials is at or below 0.1%. In structures of at least 20 years age,  $D_a$  is generally in the range  $0.1 - 1 \cdot 10^{-12} \text{ m}^2/\text{s}$ , with lower values for blast furnace slag (and fly ash) cement concrete and higher values for Portland cement concrete [8]. If significantly different values are found, the analysis needs to be checked or the sampling needs to be repeated.

Corrosion initiation is assumed to occur when the critical chloride content has reached the steel. Considerable scatter has been found for this parameter in the laboratory [13, 14] and in the field [15]. The estimated mean value for the critical chloride content is 0.5% by mass of binder. This value is based on over 100 observations of presence or absence of corrosion at a wide range of chloride contents in a 70-year-old Portland cement concrete tunnel in the Netherlands exposed to de-icing salt. In or close to cores taken for chloride analysis, corrosion had occurred where 0.5% or more of chloride was present; no corrosion was present below 0.5%. The threshold is taken independent of cement type, as is justified by a recent literature overview showing that the effect of binder type on the threshold is relatively small [16].

The assumed  $C_{crit}$  value may be too low for a case at hand. This can be inferred if the modelling results suggest that corrosion should have been already initiated, while this is obviously not the case from visual observation (absence of corrosion damage and rust staining) or by other means, e.g. potential mapping. Of course, in all cases where no corrosion is obvious, the assumed threshold may be too low. The procedure is conservative with respect to such a situation, which was deemed acceptable. In the Recommendation, additional testing is suggested as an option if the assumed  $C_{crit}$  is apparently incorrect, following a pragmatic procedure based on determining chloride content at rebar level in (additional) cores with reinforcing bars, which is not further described here for the sake of brevity. After publication of the Recommendation, a more elaborate and time consuming but probably more accurate procedure was published based on exposure of cores from mature structures [17].

Subsequently, the progress of chloride transport is predicted until the assumed critical chloride content is reached at the mean depth of the steel, which is thought to occur at time  $t_i$ . The same basic equation is used, now with inputs  $C_s$ ,  $C_i$  and  $D_a$  from the fitting of equation (1) and  $x_m$  from the cover depth measurements for each Test Area:

$$C_{crit} = C_s - (C_s - C_i) \cdot \text{erf} \left( \frac{x_m}{\sqrt{4 \cdot D_a \cdot t_i}} \right) \quad (2)$$

The outcome from this calculation is  $t_i$ , the expected age at corrosion initiation. The remaining time until initiation then is that age minus the age at inspection:  $t_m = t_i - t_{insp}$ . For each Test Area, six results for  $t_m$  are obtained. For

further interpretation of these results, how to handle uncertainty needs to be considered first.

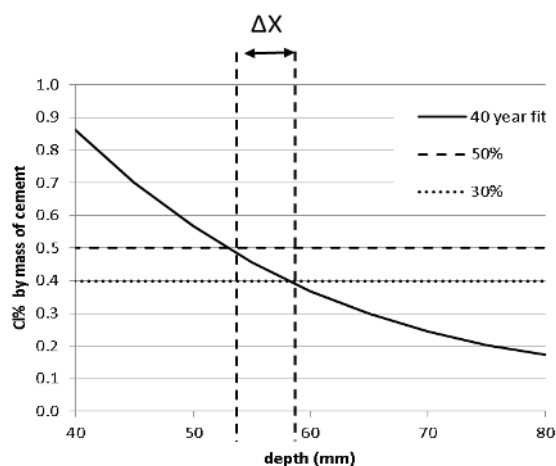
## 5 Uncertainty and further interpretation

The result for the remaining time to corrosion initiation obtained using equation 2,  $t_m$ , will contain considerable uncertainty due to the inevitable variation in parameters (chloride load, concrete chloride penetration resistance, cover depth) and the uncertainties in the critical content and the model (e.g. assuming  $C_s$  and  $D_a$  constant). In statistical terms, the result  $t_m$  for each core is a deterministic value and consequently the predicted event of corrosion initiation at time  $t_m$  will have a probability of failure (occurrence) of 50%, which appears too high for general application. For example, preventive measures such as hydrophobic treatment should be taken well in advance of the moment of initiation to be effective [18]. In addition, it may take several years before protection measures are carried out; in the meantime, corrosion related damage may already have developed, which causes the need for additional work on repairs. On the other hand, a probability of failure of 10% that is used in quantitative service life design methods for new structures [4, 5, 6] seems too low, because various uncertainties that need to be considered in the design and execution phases are no longer relevant; moreover existing structures can be inspected and tested. Consequently, the committee has chosen a target probability of failure of 30%. Such a lower probability than 50% can be obtained by taking lower-than-mean values (e.g. characteristic values) for one or more parameters.

Out of multiple options, applying a safety margin on the concrete cover was preferred, similar to the approach taken in a CUR Guideline for design of new structures [6]. The principle is illustrated in Figure 1. In this calculated profile, the chloride content at 53 mm depth is 0.5%, which is equal to the (assumed) critical content, so a rebar at that depth has a 50% probability of initiating corrosion. The chloride content at (a rebar at) 58 mm depth is 0.4%, which corresponds to a lower than 50% probability of corrosion. A series of probabilistic calculations for a representative case using TNO's software Prob2B® showed that with typical mean values and standard deviations for the input parameters, a probability of failure of 32% was found for a hypothetical bar at 5 mm higher cover depth than for a bar that was at the 0.5% chloride content boundary. This provides a means to obtain a reduced failure probability at mean bar depth: by the time 0.5% chloride has arrived at a depth that is 5 mm less than the mean cover depth, the chloride content at the mean bar depth is such that it will initiate corrosion with 32% probability.

Based on this result, the methodology works as follows. A safety margin  $\Delta x$  of 5 mm is subtracted from the mean concrete cover depth  $x_m$ , and the remaining time-to-corrosion (arrival of 0.5% chloride) is calculated for  $x_m - 5$  mm. This is the time to corrosion initiation with a probability of failure of about 30%, which is called  $t_R$ . Only if the cover depth for the Test Area has a high

variability, expressed by a standard deviation of more than 10 mm,  $\Delta x$  should be given the value of half the actual standard deviation of the cover depth.



**Fig. 1.** Illustration of the principle of reducing the cover depth by a safety margin  $\Delta x$

The following serves to further illustrate the effect of the safety margin. The hypothetical case is a 40-year-old bridge exposed to de-icing salt (XD3). The chloride profiles can be described by  $C_s = 1.5\%$ ,  $C_i = 0.1\%$ ,  $D_a = 0.35 \cdot 10^{-12} \text{ m}^2/\text{s}$ ;  $C_{crit}$  is 0.5%. Standard deviations are assumed to be 0.4%, 0.025%,  $0.07 \cdot 10^{-12} \text{ m}^2/\text{s}$  and 0.2%, respectively. The cover depth is variable with a standard deviation of 10 mm. All parameters were assumed to have a normal distribution. With this input, the failure probability at 40 years was calculated for various cover depths, see Table 1. It shows that a safety margin of 5 mm (37 mm cover depth instead of 32 mm) reduces the probability of failure from 50% to 32%.

**Table 1.** Failure probability for initiation of corrosion at 40 year age for hypothetical case

mean cover depth	32 mm	37 mm	42 mm	50 mm
failure probability	50%	32%	21%	10%
safety margin	0 mm	5 mm	10 mm	18 mm

Further interpretation for a Test Area proceeds as follows. The time-to-corrosion for the mean cover depth  $t_m$  and for the case with the safety margin  $t_R$  are calculated for all (at least six) cores. The  $t_m$  results are for illustrative purposes. Next, all results for  $t_R$  are classified in three intervals: A) equal to or less than 5 years, B) 5 to 15 years and C) more than 15 years, as shown in Table 2. The shortest interval in which at least three of those classifications fall, is considered the final result for the test area. Three or more out of six with a preference for the shortest interval is seen as a best representative result, without being overly conservative. If two cores fall in each class, the overall result should be taken as  $\leq 5$  years; or more cores need to be taken and investigated; or expert judgement should be applied, considering specific core locations, possible physical

causes and the structural importance of the element. Examples for classification of six results are given in Table 3.

**Table 2.** Classification of results from individual cores in intervals

Remaining time to corrosion initiation $t_R$ per core (includes safety margin)	Classification
$\leq 5$ year	A
5 – 15 year	B
$> 15$ year	C

**Table 3.** Examples of classification of the results of six cores from a Test Area

Classifications for individual cores	Test Area Classification
AAA B CC	A
BBB CCC	B
AA BB CC	A, additional testing or expert judgement

## 6 Case study

### 6.1. Background

The highway section A9 Holendrecht-Diemen, located southeast of Amsterdam, has recently been expanded to keep the region accessible. The highway was widened from 2 x 3 lanes to 2 x 5 lanes. The project was a so called DBFM (Design, Build, Finance and Maintain) contract and was carried out by a consortium of companies. After finishing the project, the consortium will be responsible for maintenance over the next 20 years. The consortium wanted to determine the remaining life of several concrete bridges in the highway. The bridges were all approximately 40 years old. The required remaining service life equals 30 years. A preliminary version of the procedure outlined above has been applied to several bridges in this project, one of which is described here as an example.

### 6.2. Structure description and methods applied

The structure (KW15) consists of a northern and a southern bridge. The decks consist of one post-tensioned concrete slab each with a total length of 45 m in two spans. Each deck is supported by two abutments and two mid-span piers with two columns each, see Figure 2.

The remaining life of the bridges was investigated using the methods described above. The focus of the investigation was on the abutments and the bridge decks, being the main critical parts. The first step was a desk study of existing inspection reports in order to identify:

- defects without visible damage (leaking joints)
- visible damage to the concrete,
- repaired spots possibly indicating durability issues in the past.

The second step was a visual inspection to verify the results of the desk study and assess new damage, if any. The third step was to drill cores to determine chloride profiles and carbonation depths. Around each core

location, the cover depth was measured using non-destructive testing on at least six bars. Six cores (Ø 65 mm) per structural member and per relevant side were drilled for chloride testing. Cores for carbonation testing were drilled from the front vertical side of the abutments and the bottom side of the bridge decks.



Fig. 2. Side view (top) of investigated bridge and columns supporting the decks (bottom)

### 6.3. Results

The bridges did not show any concrete damage (other than small spots of collision damage) which was in line with the desk study. At both abutments strong leakage occurred due to dysfunctional drains.

The mean cover depth around the core locations varied between 25 and 45 mm. Cores for chloride profiles were drilled from the front vertical side of the abutments and from the top side of the bridge decks (through the asphalt). The decks have a slight inclination for water discharge; the cores were drilled at the low side. Examples of the obtained chloride profiles and fitted curves are shown in Figures 3 and 4. The complete chloride profiles from the top of the deck could be well described by the diffusion equation. In the profiles of the abutments the first one or two points needed to be discarded in the fit.

A visual assessment of the cores showed that blast furnace slag cement was used for all investigated parts (indicated by a dark blue colour, as opposed to grey for Portland cement). No indication was found that other degradation mechanisms than chloride ingress and carbonation are relevant for the remaining service life.

The compressive strength was determined on separate cores (Ø95 mm) following EN 13791. The obtained strength class was quite high: C70/85 for the decks and C55/67 for the abutments.

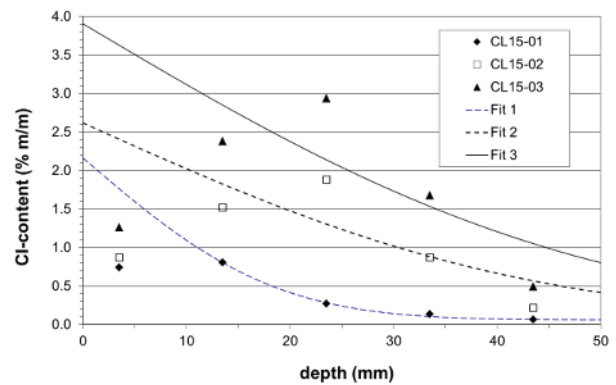


Fig. 3. Measured chloride profiles (symbols) and best fits (curves), Northern abutment

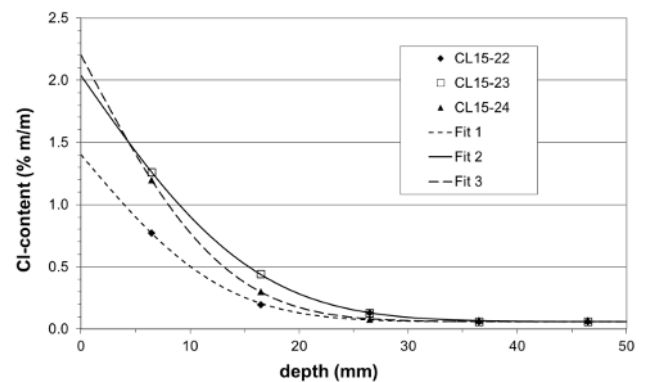


Fig. 4. Measured chloride profiles (symbols) and best fits (curves), top side of deck

The remaining time to chloride induced corrosion initiation was calculated for all cores using equation 2, the model parameters obtained by fitting each chloride profile and the average measured cover depth, with ( $t_R$ ) and without ( $t_m$ ) the safety margin  $\Delta x$  of 5 mm. Tables 4 to 6 show the results. The initial chloride content was about 0.06% in all cores.

Table 4. Fitting results of six cores, remaining time to corrosion initiation and intervals, abutments Northern bridge

Parameter (Unit)	CL-01	CL-02	CL-03	CL-04	CL-05	CL-06
$C_s$ (%)	2.16	2.62	3.91	1.57	6.99	3.45
$D_a$ (10 <sup>-12</sup> m <sup>2</sup> /s)	0.09	0.48	0.62	0.12	0.20	0.41
$x_m$ (mm)	29	45	42	42	35	34
$t_{R\Delta x=5}$ (year)	25	-11	-25	> 100	-18	-25
$t_{m\Delta x=0}$ (year)	55	-3	-21	> 100	-11	-20
Interval	C	A	A	C	A	A

**Table 5.** Fitting results of six cores, remaining time to corrosion initiation and intervals, abutments Southern bridge

Parameter (Unit)	CL-13	CL-14	CL-15	CL-16	CL-17	CL-18
Cs (%)	4.25	3.13	0.99	0.58	5.73	0.53
Da (10-12 m <sup>2</sup> /s)	0.13	0.09	0.08	0.19	0.16	0.22
xm (mm)	34	29	33	30	45	41
t <sub>R</sub> Δx=5 (year)	0	8	> 100	> 100	12	> 100
t <sub>m</sub> Δx=0 (year)	15	30	> 100	> 100	25	> 100
Interval	A	B	C	C	B	C

**Table 6.** Fitting results of six cores, remaining time to corrosion initiation and intervals, deck Northern bridge

Parameter (Unit)	CL-19	CL-20	CL-21	CL-22	CL-23	CL-24
Cs (%)	0.95	1.27	1.43	1.40	2.04	2.20
Da (10-12 m <sup>2</sup> /s)	0.06	0.19	0.20	0.04	0.07	0.04
xm (mm)	35	35	45	25	25	25
t <sub>R</sub> Δx=5 (year)	> 100	52	89	> 100	26	60
t <sub>m</sub> Δx=0 (year)	> 100	85	> 100	> 100	62	> 100
Interval	C	C	C	C	C	C

The t<sub>R</sub> values from the bridge decks are all >30 years except one. The t<sub>R</sub> values from the abutments are much lower and even negative for four cores from the northern viaduct. The t<sub>m</sub> values are also negative in these cases. This implies that corrosion should have been initiated in the past. Nevertheless, damage by corrosion has not (yet) been observed. Possibly, the critical chloride content for this concrete is higher than the assumed value of 0.5%, or this is due to the relatively poor fit of the profiles (Figure 3), or it is related to the relatively high carbonation depths (see below). Values found for C<sub>s</sub> in the deck were in the expected range (1-2%), but in the abutments some were higher (3% or more), either due to a high chloride load from leakage or possibly again due to the relatively poor fit. D<sub>a</sub> values between 0.05 and 0.6 \* 10<sup>-12</sup> m<sup>2</sup>/s corresponded to typical values for mature blast furnace slag cement concrete [8]. The apparent absence of corrosion in the profiles with higher chloride ingress could be due to favourable aspects of the steel/concrete interface (SCI) such as the steel type or the slag cement. In particular various influences of the SCI on corrosion initiation are insufficiently understood at present and require more research [15, 16].

Carbonation depths were relatively low, with one exception. In the northern abutment of the southern bridge the carbonation depths almost equal the cover depths. Carbonation might be determining the service

life for this part, or the combination with chloride penetration. Without going into detail about this issue, carbonation of blast furnace slag cement concrete has implications for chloride penetration resistance [19] and quite possibly also for the critical chloride content [20].

## 7 Conclusions

In a recent CUR Technical Recommendation [7], a procedure is described for determining the time-to-corrosion initiation for reinforced concrete elements exposed to de-icing salts. It applies to existing reinforced and prestressed civil engineering structures of ten and preferably at least 20 years old. Based on measuring the actual cover depth, analysing chloride profiles, additional parameters and a simple diffusion model, the expected time to chloride induced corrosion initiation for a particular structural element can be predicted. Uncertainties are taken into account by applying a safety margin to the mean cover depth. Results of at least six cores from each test area are classified in three ranges: five years or less, five to fifteen years, or more than 15 years. In the document, a similar approach is described for carbonation induced corrosion. The resulting estimated time to corrosion initiation provides a lower boundary for the remaining service life. At present, quantifying the loss of steel section during the propagation phase is not possible with sufficient accuracy.

With the present state of knowledge, this procedure provides the best possible approximation of the expected time to corrosion initiation for application in practical cases; results are not claimed to be highly accurate, which is borne out in the classification of results. It is hoped and expected that using this procedure, useful new experience will be gained and significant improvement can be made in the future. Results from an example case show that the procedure works rather well.

The road extension of the A9 Gaasperdammerweg was performed by IXAS, a consortium of Ballast Nedam, Heijmans, Fluor and 3i. IXAS is acknowledged for providing the results of the service life investigation of KW15. Contributions of SBRCURnet committee 2140 members are greatly appreciated.

## References

1. L. Bertolini, B. Elsener, P. Pedferri, E. Redaelli, R. Polder, (2013). *Corrosion of Steel in Concrete: Prevention, Diagnosis, Repair*, (Wiley-VCH, Weinheim, 2013)
2. G. Gaal, Prediction of Deterioration of concrete bridges, Ph.D. thesis, Delft University Press, Delft (2004)
3. R. Polder, W. Peelen, W. Courage, *Materials and Corrosion* **63** (12) 1147-1153 (2012)
4. DuraCrete, Final Technical Report, CUR, Gouda (2000)
5. fib, Model Code for Service Life Design, fib Bulletin 34 (2006)

6. G. van der Wegen, R. Polder, K. van Breugel, *HERON* **57** (3) 153-168 (2012)
7. CUR, Aanbeveling 121, Bepaling van de ondergrens verwachte restlevensduur van bestaande gewapende betonconstructies, in Dutch, SBRCURnet/CROW, Ede (2018)
8. R. Polder, M. de Rooij *HERON*, **50** (3) 133-143 (2005)
9. R. Vergoossen, M. Boutz, Proceedings of the joint IABSE-fib Conference 'Codes in Structural Engineering. Developments and Needs for International Practice', Dubrovnik, 1217-1222 (2010)
10. U. Angst, R. Polder, *CCR*, **56**, 40-51 (2014)
11. J. Gulikers, Testing and Modelling the Chloride Ingress into Concrete, *RILEM PRO* **19**, 439-450 (2001)
12. M. Collepardi, A. Marcialis, R. Turriziani, *J.Am.Cer.Soc.* **55**, 534-535 (1972)
13. U. Angst, B. Elsener, C.K. Larsen, Ø. Vennesland, *CCR* **39**, 1122-1138 (2009)
14. J. Pacheco, R. Polder, *HERON* **61** (2) 99-119 (2016)
15. U. Angst, M. Geiker, A. Michel, C. Gehlen, H. Wong, B. Isgor, B. Elsener, C. Hansson, R. François, K. Hornbostel, R. Polder, M. Alonso, M. Sanchez, M. Correia, M. Criado, A. Sagüés, N. Buenfeld, *MAAS*, **50** (2) 143 (2017)
16. U. Angst, M. Geiker, M. Alonso, R. Polder, B. Isgor, B. Elsener, H. Wong, A. Michel, K. Hornbostel, C. Gehlen, R. François, M. Sanchez, M. Criado, H. Sorensen, C. Hansson, R. Pillai, S. Mundra, J. Gulikers, M. Raupach., J. Pacheco, A. Sagüés, *MAAS* **52**, 88 (2019)
17. U. Angst, C. Boschmann, M. Wagner, B. Elsener, J. Visualized Experiments, 126, (2017)
18. R. Polder, H. Borsje, J. de Vries, *HERON* **46** (4) 227-238 (2001)
19. P. Holthuisen, O. Copuroglu, R. Polder, fib symposium, Maastricht, 73-82 (2017)
20. Y. Segui, U. Angst, B. Elsener, *RILEM Technical Letters* **2** 59-66 (2017)