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A METHOD FOR DETERMINING THE REMAINING TIME TO CHLORIDE INDUCED CORROSION INITIATION OF EXISTING CONCRETE STRUCTURES

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Abstract

Owners of concrete structures would benefit from knowing when to expect corrosion initiation in a particular structure. Presently, no accepted procedures for testing existing structures for the remaining time to corrosion initiation are available. This paper proposes such a procedure, based on our experience and additional considerations. From about 20 years age, existing structures contain the concrete's response to actual environmental loads, e.g. chloride ingress profiles. By measuring the actual cover depth, taking chloride profiles, assuming a few parameters and a simple model, the expected time to corrosion initiation for a particular test area can be predicted. Numbers of cores and samples per core are given. Uncertainties can be taken into account by applying a calculatory reduction of the cover depth. Results of at least six cores per test area are classified and suggested 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 is issued early 2018. It was applied to 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 Technical Recommendation setup by SBRCURnet committee 2140 “Remaining service life of existing concrete structures”, published early 2018. The procedure is based on existing practice of a number of persons and organisations with condition oriented inspection, sampling and interpretation for existing concrete structures, among others [7]. The procedure takes both carbonation and chloride induced corrosion into account. It is also intended to address 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.

2. BASIC PRINCIPLES

Basic considerations are as follows. Existing structures have a time-independent distribution of cover depth. If they are exposed to de-icing salts, 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 penetration), critical chloride content, moisture distribution and carbonation, all with a more or less fixed spatial distribution, which for simplicity can be assumed fixed in time. Cover depths and chloride penetration profiles can be measured relatively easily. However, sample size and frequencies need to be considered due to significant spatial variation resulting from execution, environmental loads and inherent to the presence of aggregate [8]. Values for the critical chloride content can be based on experience or can be tested following a simplified pragmatic approach. Simplified models for chloride transport and carbonation are provided. 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. The proposed procedure contains considerable simplifications and assumptions, which are dealt with by applying a safety margin to the mean cover depth.

3. STEPS TO BE TAKEN

The following steps have to be taken when a structure is investigated:

- 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. 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 test areas (TA).

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 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, or;
- 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 [7], 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 (or less if low ingress is expected)
- each slice is dissolved in acid, the residue weighed and chloride 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 [9]
- (optional) cement type is determined, visually or using thin section microscopy.

For each core, a chloride penetration profile is found for which a diffusion profile [10] is fitted using the least squares method to:

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

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_{insp} (s); C_s chloride surface content (%); C_i initial chloride content (%); erf the error function; D_a the apparent chloride diffusion coefficient (m^2/s). The input is C(x,t) 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. 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 1-3%, C_i in concrete made using non-contaminated raw materials at or below 0.1%. In structures of at least 20 years age, D_a is generally in the range $0.1 – 1 * 10^{-12}$ m^2/s, with lower values for blast furnace slag (and fly ash) cement concrete and higher values for Portland cement concrete [7]. 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 [11,12] and in the field [13]. The assumed mean value for the critical chloride content is 0.5%, 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.

Additional testing is suggested as an option if the assumed Ccrit is apparently incorrect, which is not further described here.

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, D_a from the fitting of equation (1) and x_m from the cover depth measurements for each Test Area:

$$C_{crit}=C_s-(C_s-C_i).\mathrm{erf}\left(\frac{x_m}{\sqrt{4.D_a.t_I}}\right) \quad (2)$$

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: $t_m = t_I - t_{insp}$. Testing six cores, six results are obtained. For further interpretation of these results, how to handle uncertainty needs to be considered.

4. UNCERTAINTY AND FURTHER INTERPRETATION

The result for the remaining time to corrosion initiation obtained using equations 1 and 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 (assuming constant C_s and D_a). In statistical terms, the result (for each core) is a deterministic value, so it will have a probability of failure of 50%, which appears too high for general application. 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 in the design and execution phases are no longer relevant, and existing structures can be inspected. Consequently, a target probability of failure of 30% was chosen. Such a lower probability can be obtained by taking lower-than-mean values (e.g. characteristic) for one or more parameters. Out of multiple options, applying a safety margin, Δx, on the concrete cover was preferred. The methodology works as follows. A safety margin Δx of 5 mm is subtracted from the mean concrete cover depth, x_m, and the remaining time-to-corrosion is calculated for x_m-5 mm, producing t_R. This safety margin was justified by probabilistic calculations made using TNO's software Prob2B® for a representative case. For typical mean and standard deviations for the input parameters, a probability of failure of 32% was found for Δx = 5 mm. Only in case the cover depth has a high variability, expressed by a standard deviation of more than 10 mm, Δx should be given the value of half the actual standard deviation of the cover for the Test Area.

Further interpretation for a test area proceeds as follows. The time-to-corrosion for the mean input values t_m and for the case with the safety margin t_R are calculated for all (at least six) cores. The mean result is 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 1. 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 ≤ 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 are given in Table 2.

Table 1 Classification of results from individual cores in intervals

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

Table 2 Examples of classifications per test area

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

5. CASE STUDY

5.1 Background

The highway section A9 Holendrecht-Diemen, located southeast of Amsterdam, is currently being expanded to keep the region accessible. The highway will be widened from 2 x 3 lanes to 2 x 5 lanes. The project is a so called DBFM (Design, Build, Finance and Maintain) contract and is carried out by a consortium of companies. After finishing the project, the consortium is responsible for maintenance over the next 20 years. The consortium wants to determine the remaining life of several concrete bridges in the highway. The bridges are 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, one of which is described here as an example.

5.2 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 piers with two columns each, see Figures 1 and 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 a) defects without visible damage (leaking joints) b) visible damage to the concrete, c) 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. At each core location the cover depth was measured using non-destructive testing. Per structural member and per relevant side six cores (Ø 65 mm) were drilled, both for chloride and carbonation.

Figure 1 Side view (left) of investigated bridge and columns supporting the decks (right)

5.3 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 cover depth varied from 25 to 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 and the cores were drilled at the low side. Examples of the obtained chloride profiles and fitted curves are shown in Figures 2 and 3. 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.

Cores for carbonation testing were drilled from the front vertical side of the abutments and the bottom side of the bridge decks. The carbonation depth was generally < 20 mm. Only in the northern abutment of the southern viaduct higher carbonation depths (28 mm) were found.

A visual assessment of the cores showed that blast furnace slag cement was used for all investigated parts. 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.

The remaining time to chloride induced corrosion initiation was calculated for all cores using eq. 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 Δx. The initial chloride content was 0.06% in all cores. Tables 3 to 5 show the results.

The t_R values from the bridge decks are all >30 years except one. The t_R values from the abutments are much lower and even negative for four cores from the northern viaduct. The t_m 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 is higher than 0.50% for this concrete, or this is due to the relatively poor fit of the profiles (Figure 2 left), or it is related to the relatively high carbonation depths (see below).

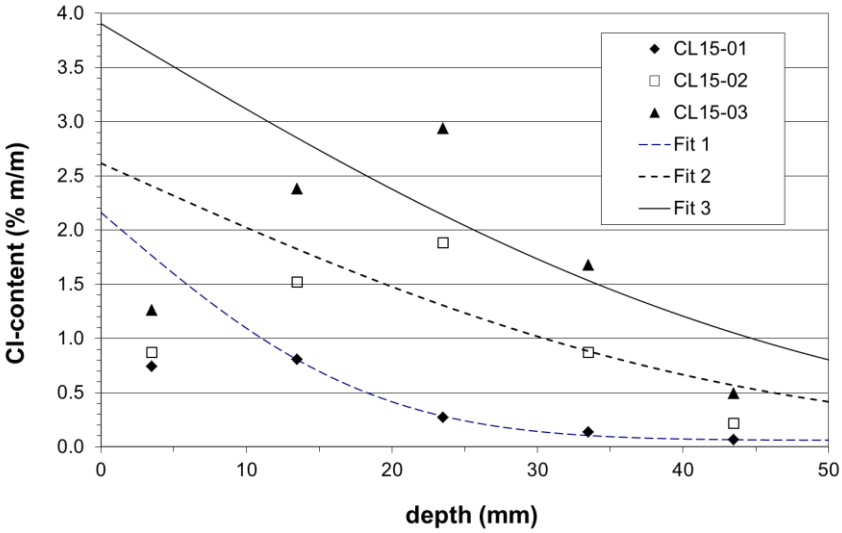

Figure 2 Measured chloride profiles (symbols) and best fits (curves), Northern abutment

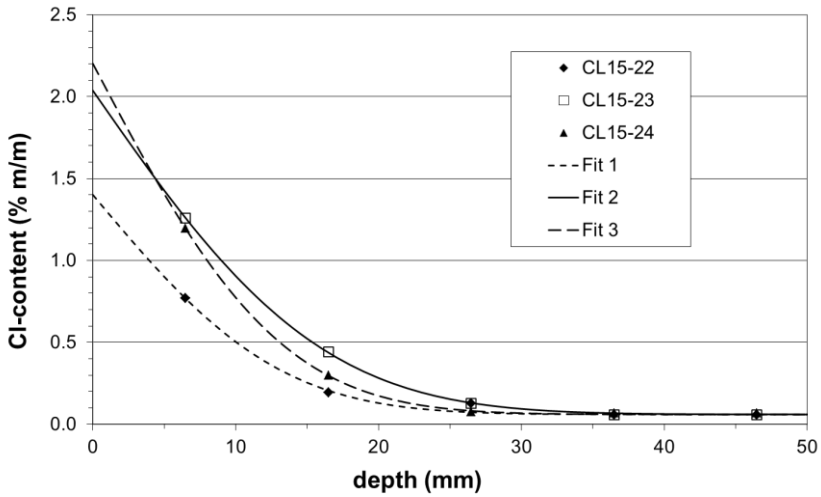

Figure 3 Measured chloride profiles (symbols) and best fits (curves), top side of deck

Table 3 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	($10^{-12}m^2/s$)	0.09	0.48	0.62	0.12	0.20	0.41
x_m	(mm)	29	45	42	42	35	34
t_R (Δx=5)	(year)	25	-11	-25	> 100	-18	-25
t_m (Δx=0)	(year)	55	-3	-21	> 100	-11	-20
Interval		C	**A**	**A**	C	**A**	**A**

Table 4 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
C_s	(%)	4.25	3.13	0.99	0.58	5.73	0.53
D_a	(10^{-12} m^2/s)	0.13	0.09	0.08	0.19	0.16	0.22
x_m	(mm)	34	29	33	30	45	41
t_R (Δx=5)	(year)	0	8	> 100	> 100	12	> 100
t_m (Δx=0)	(year)	15	30	> 100	> 100	25	> 100
Interval		A	B	**C**	**C**	B	**C**

Table 5 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
C_s	(%)	0.95	1.27	1.43	1.40	2.04	2.20
D_a	($10^{-12}m^2/s$)	0.06	0.19	0.20	0.04	0.07	0.04
x_m	(mm)	35	35	45	25	25	25
t_R (Δx=5)	(year)	> 100	52	89	> 100	26	60
t_m (Δx=0)	(year)	> 100	85	> 100	> 100	62	> 100
Interval		**C**	**C**	**C**	**C**	**C**	**C**

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 [14] and quite possibly also for the critical chloride content [15].

5. CONCLUDING REMARKS

In a recent Dutch Technical Recommendation, 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, taking chloride profiles, additional parameters and a simple diffusion model, the expected time to chloride induced corrosion initiation for a particular 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. 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, the proposed 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.

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