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# Use of chloride ingress model for condition assessment in bridge management

Authors:

**Ivan Zambon**, PhD. CE
University of Natural Resources and Life Sciences
Department of Civil Engineering and Natural Hazards
ivan.zambon@outlook.com

**Anja Vidović**, PhD. CE
University of Natural Resources and Life Sciences
Department of Civil Engineering and Natural Hazards
anja.vidovic@boku.ac.at

Assoc.Prof. **Alfred Strauss**, PhD. CE
University of Natural Resources and Life Sciences
Department of Civil Engineering and Natural Hazards
alfred.strauss@boku.ac.at

Assist.Prof. **Jose Matos**, PhD. CE
Minho University
Civil Engineering Department
jmatos@civil.uminho.pt

Original scientific paper

Ivan Zambon, Anja Vidović, Alfred Strauss, Jose Matos

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Although there is a large number of established Bridge Management Systems worldwide, there seems to be a lack of those that utilise the knowledge on material properties and environmental loading. Hence, as a supplement to current practice, the paper examines the use of the chloride ingress model, supported by the International Federation for Structural Concrete, for preliminary assessment in bridge management. The focus is set on analysing application of the model in realistic situations on existing concrete bridges for which the information on material and environmental properties is lacking.

**Key words:**
chloride ingress, bridge management, deterioration, durability, preliminary assessment, concrete bridges

Izvorni znanstveni rad

Ivan Zambon, Anja Vidović, Alfred Strauss, Jose Matos

### Primjena modela prodora klorida za ocjenu stanja u gospodarenju mostovima

Iako se danas diljem svijeta primjenjuju brojni sustavi za gospodarenje mostovima, čini se da ipak nema dovoljno sustava u kojima bi se koristilo znanje o svojstvima materijala i opterećenju iz okoliša. U ovom se radu, kao doprinos sadašnjoj praksi, analizira primjena modela prodora klorida, što podržava Međunarodni savez za betonske konstrukcije, a koristi se za preliminarno ocjenjivanje u okviru gospodarenja mostovima. Prije svega, analizira se primjena modela u realnim situacijama za ocjenu postojećih betonskih mostova za koje ne postoje podaci o materijalima i svojstvima okoliša.

**Ključne riječi:**
prodor klorida, gospodarenje mostovima, propadanje, trajnost, preliminarno ocjenjivanje, betonski mostovi

Wissenschaftlicher Originalbeitrag

Ivan Zambon, Anja Vidović, Alfred Strauss, Jose Matos

### Anwendung des Penetrationsmodells von Chlorid bei der Beurteilung des Zustands beim Brückenmanagement

Obwohl heute weltweit zahlreiche Brückenmanagementsysteme angewendet werden, scheint es jedoch nicht ausreichend Systeme zu geben, in denen die Kenntnisse über die Materialeigenschaften und die Umweltbelastung angewendet werden. In dieser Abhandlung wird als Beitrag zur aktuellen Praxis die Anwendung des Penetrationsmodells von Chlorid analysiert, was vom Internationalen Verband für Betonkonstruktionen unterstützt wird, und wird als vorläufige Beurteilung im Rahmen des Brückenmanagements angewendet. Zunächst wird die Anwendung des Modells unter realen Bedingungen für die Beurteilung der bestehenden Betonbrücken analysiert, zu denen Daten über die Materialien und die Umwelteigenschaften vorliegen.

**Schlüsselwörter:**
Penetration von Chlorid, Brückenmanagement, Verfall, Dauerhaftigkeit, vorläufige Beurteilung, Betonbrücken

## 1. Introduction

During operation, the predetermined service life of structures is challenged by various load effects, ageing, and environmentally caused degradation. Hence, owners and operators of existing structures face complex decisions regarding maintenance and repair strategies or possible replacements [1]. An increasing number of computer-aided Bridge Management Systems (BMSs) has been developed over the past two decades in order to assist engineers in the analysis of a large number of bridges, and in forming appropriate decisions [2]. A properly assessed structural condition presents itself as one of crucial segments in the development of decision strategies. Since a full structural analysis is expensive and time consuming, current bridge management practices mostly rely on the outcome of visual inspections as the main means for preliminary condition assessment.

The existing standards and codes are mainly oriented toward design of new structures, while only recently the attention has been shifted to the development of international standards and codes for the assessment of existing structures. The most significant of these is the international standard ISO 13822: Bases for design of structures – Assessment of existing structures [3], which divides the condition assessment procedure into the following steps:

- the specification of the assessment objective,
- defining possible scenarios,
- preliminary assessment,
- detailed assessment,
- reporting the results, and
- repetition of the sequence, if necessary.

In addition, the preliminary assessment comprises:

- study of documents and other evidence,
- preliminary inspection,
- preliminary checks,
- decision on immediate actions, and
- recommendation for detailed assessment.

The decision on whether an immediate action or further investigation is necessary or not should be made based on the study of documents and other evidence, and according to qualitative grading of structural conditions obtained from preliminary inspection.

In addition to verification of current condition according to the Model Code for Concrete Structures [4], a prognosis of future performance should be made based on condition assessment. Therefore, one of main BMS components should be to predict long-term performance using a condition prediction model.

In 2008, the International Association for Bridge Management and Safety (IABMAS) prepared a questionnaire aimed at collecting the BMS data from all over the world [5]. Since then, three reports on bridge management systems have been published: in 2010, 2012, and 2014. Twenty-five different BMSs were analysed in the last report from 2014. Mirzaei, *et al.* [6] reported that nineteen analysed BMSs can predict deterioration and subsequent duration of service life. However, a lack of BMSs that utilise the knowledge on how a bridge is affected by surrounding environment can be noticed. Likewise, while most of them utilize the Markov chain theory for condition prediction, very few take into account actual specific material properties of every bridge [7].

Over the last decades, numerous researchers and organizations have invested substantial efforts in developing and enhancing the knowledge pool on analytical models based on actual physical and chemical damaging processes involved in deterioration, where the chloride ingress has been one of the most explored processes. Driven by recent development in the area of deterioration models, and by the need for incorporating effects of material and environmental properties in BMSs, this paper aims at examining the use of the chloride ingress model in preliminary assessment of a large number of concrete bridges forming part of transport infrastructure networks.

## 2. Chloride ingress

Reinforcement corrosion is undoubtedly one of the most frequent limiting factors for the service life of reinforced concrete structures [8]. A microscopically thin oxide layer, which forms on the surface due to alkalinity of the surrounding concrete, keeps steel bars passive. If alkalinity of concrete is lost due to penetration of chlorides or by carbonation, the protective layer dissolves, and corrosion can be triggered [9].

In most operational standards that are currently in use, the design associated with durability is based on the deemed-to-satisfy approach, rather than on other safety formats such as the fully probabilistic design format or the partial safety factor format. In this way, by following a set of predetermined rules, it is ensured that the target reliability for not violating the relevant limit state during the design service life is not exceeded, when the concrete structure or component is exposed to design situations. In that manner, the minimum concrete cover is defined in EN 1992-1-1:2004 [10] as a combination of structural and exposure classes, which comprise different environmental conditions that cause deterioration. Descriptions and several informative examples for each class can be found in EN 206-1 [11]. The environmental conditions causing chloride ingress are divided into the XS category (4 classes) and the XD category (3 classes). The XS category represents corrosion induced by chlorides from seawater, while the XD category represents corrosion induced by chlorides other than those originating from seawater.

Contrary to the deemed-to-satisfy approach, many different approaches have so far been developed for modelling the time-dependent process of chloride ingress in concrete. The model employed in this article is supported by the *fib* in Model Code 2010 [4], Bulletin 34 [12], Bulletin 59 [13], and Bulletin 76 [14]. It is based on the Fick's 2nd law of diffusion, which states that the transport of chlorides in concrete is mainly diffusion-controlled. The Crank's solution [15] for the Fick's second law was first applied by Collepardi *et al*. [16], resulting in a prediction model based on error function. A fully probabilistic design approach for modelling the chloride-induced depassivation in uncracked concrete is based on the limit state equation, Eq. (1), in which the critical chloride content $C_{crit}$ is compared to the actual chloride content $C(c,t)$ at the depth of reinforcing steel.

$$g(c,t) = C_{crit} - C(c,t) \tag{1}$$

where:

$C_{crit}$ - the critical chloride content [wt.-%/c]
$C(c,t)$ - the actual chloride content at the depth of reinforcing steel [wt.-%/c].

The content of chlorides at a corresponding depth $x$ is given by Eqs. (2) to (4):

$$C(x,t) = C_0 + (C_{S,\Delta x} - C_0)\cdot\left[1 - erf\,\frac{x-\Delta x}{2\cdot\sqrt{D_{app}(t)\cdot t}}\right] \tag{2}$$

$C_0$ - the initial chloride content [wt.-%/c]
$C_{S,\Delta x}$ - the chloride content at depth $\Delta x$ [wt.-%/c]
$x$ - the depth with a corresponding content of chlorides ($x = c$ – concrete cover) [mm]
$\Delta x$ - the depth of convection zone [mm]
$D_{app}(t)$ - the apparent chloride diffusion coefficient [m$^2$/s]
$t$ - the time [s].

The initial chloride content $C_0$ represents the amount of chlorides in concrete at the commencement of its use ($t = 0$). The chlorides can originate from concrete constituents or the environment (during the construction process). Although the value of initial chloride content is considered to be negligible in new structures, it could be quite high in older structures [14].

Chloride content reaches its maximum at the depth referred to as 'the convection zone', rather than on its surface, which is mainly due to a combination of dry periods and subsequent re-wettings [17]. In order to use the Fick's 2$^{nd}$ law of diffusion, the effects in the convection zone are neglected by the substitute surface $\Delta x$ and the substitute surface chloride content $C_{S,\Delta x}$. The chloride content at the substitute surface $C_{S,\Delta x}$ is determined by the material, geometrical and environmental conditions. This content can vary to a great extent. These variations are due to temporal and spatial variations in concrete humidity, frequency of application of de-icing salts, variations in chloride content of the ambient solution, etc.

The chloride migration coefficient $D_{RCM,0}$ from rapid chloride migration (RCM) test is used in this paper for quantification of the apparent chloride diffusion coefficient $D_{app}(t)$, as shown in Eq. (3).

$$D_{app}(t) = k_e \cdot D_{RCM,0} \cdot \left(\frac{t_0}{t}\right)^{\alpha} \tag{3}$$

where:

$k_e$ - the environmental variable [-]
$D_{RCM,0}$ - the chloride migration coefficient at the reference point of time [m$^2$/s]
$t_0$ - the reference point of time [y]
$\alpha$ - the ageing exponent [-].

The rapid chloride migration test is described in BUILD 492 [18], where the chloride migration coefficient $D_{RCM,0}$ is determined by means of the non-steady-state migration. The reference time $t_0$, at which the RCM is most commonly performed amounts to 28 days or, expressed in years, to 0.0767 years. The approach presents an alternative to the approach where the apparent diffusion coefficient at a reference time $D_{app}(t_0)$ is being used. This coefficient is derived from chloride profiles and/or laboratory short-term diffusion tests. It has to be noted that RCMs are methods of convenience and, since the chloride binding is not always produced to the same degree as in the diffusion test, the RCM should always be calibrated against the "chloride profiling method" (under natural conditions).

The ageing exponent $\alpha$ in Eq. (3) introduces the decrease of the chloride diffusion coefficient $D_{app}(t)$ over time and, in a mathematical context, it presents a slope in a double-logarithmic diagram. It is a parameter that is considered dependent on both material and environmental conditions. It is believed that the type of cement influences chloride diffusion through both the chloride migration coefficient $D_{RCM,0}$ and the ageing exponent $\alpha$. An illustrative example for determining $D_{RCM,0}$ and $\alpha$ according to *fib* Bulletin 76 [14] is given in Figure 1. It is shown in the left part of Figure 1 in which way $D_{app}(t)$ can be extracted for a measured chloride content $C(x,t)$ at different depths in a real structure, by fitting the curve to the Fick's 2nd law of diffusion, using non-linear regression. The actual chloride profile in the convection zone, and the length of the zone $\Delta x$, are also depicted. In the right part of Figure 1, $D_{app}(t)$ is determined for structures at different ages and, after taking into account the exposure temperature through the environmental variable $k_e$, it is plotted in the double logarithmic scale. A regression line is then drawn through the measured values of $D_{app}(t)$ obtained from field data. Furthermore, the line is forced (boundary) through mean value of measured chloride migration coefficients $D_{RCM,0}$. The whole procedure implies the use of concrete types exhibiting exactly the same properties in both RCM test as well as in profiles taken from the existing structures.

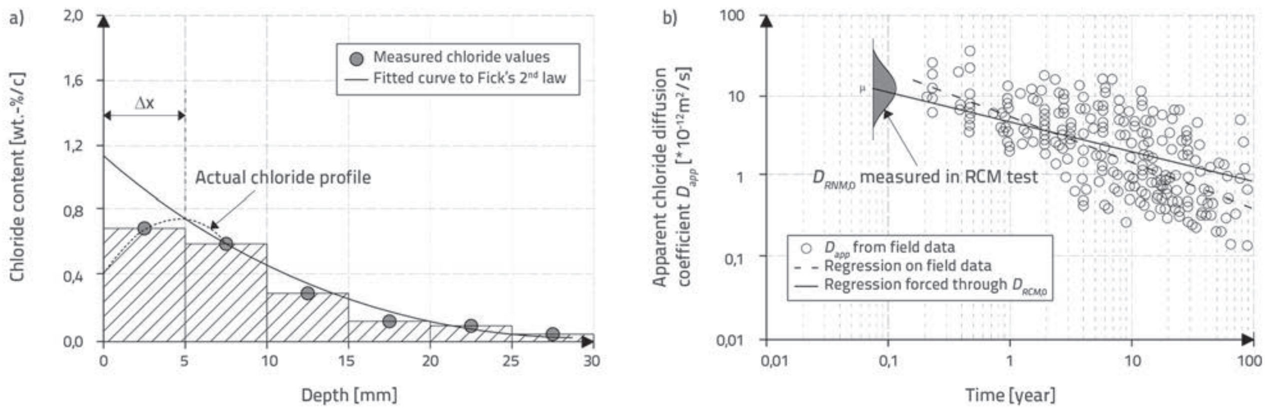


**Figure 1. Illustrative example for determining the chloride migration coefficient and ageing exponent, based on *fib* Bulletin 76 [14]**

The environmental variable $k_e$ assumes the form:

$$k_e = \exp\left[ b_e \left( \frac{1}{T_{ref}} - \frac{1}{T_{real}} \right) \right] \tag{4}$$

where:
$b_e$ - the temperature coefficient [K];
$T_{ref}$ - the reference temperature [K];
$T_{real}$ - the temperature of structural element or ambient air [K].

The transfer parameter $k_e$ includes the ambient temperature $T_{real}$, the reference temperature $T_{ref}$ = 293 [K] (for the RCM test), and the temperature coefficient $b_e$. For $T_{real}$, the average annual ambient temperature of the structure, obtained from the nearest weather station, should be used. It should be noted that, although widely used by engineers in practical applications, the model based on error function solution according to the Fick's 2nd law of diffusion still has several shortcomings. One of the most notable shortcomings is the assumption of a constant surface chloride content, which seems unlikely in chloride ingress from de-icing salts, due to the seasonal nature of loading and existence of build-up periods. The surface cover layer differs very much from the assumptions of a homogeneous, stable, non-reactive material subjected to pure diffusion. Still, the model only indirectly takes into account phenomena other than diffusion. Other shortcomings are that the model is applicable only for 1-D ingress; it does not take into account the interaction with other deterioration mechanisms, etc. A detailed overview of errors caused by application of oversimplified expressions, used in some models based on error function solution according to the Fick's 2nd law of diffusion, can be found in Tang and Gulikers [19] and Nilsson and Carcasses [20].

## 3. Case study

### 3.1. General

The Weikendorf Bridge is an Austrian Federal Railway (ÖBB) single span railway bridge that was built in 1967 in the Gänsendorf District of the state of Lower Austria. Main elements of the bridge are shown in the ground plan in Figure 2.

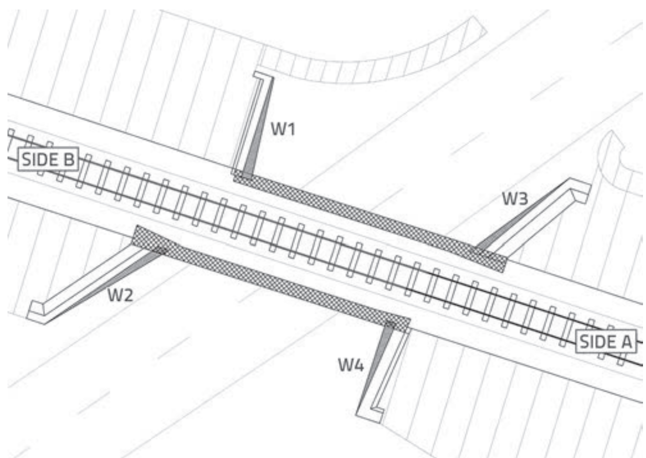


**Figure 2. The Weikendorf Bridge ground plan with main elements**

According to past inspection documents for the Weikendorf Bridge, in 2005 the bridge condition was for the first time defined as Condition State 3. Before 2005, inspection records contained only description of condition without assigning the condition state. According to *ÖBB Infrastruktur Regelwerk* [21], such condition state denotes a poor state of preservation, but with no use-related restrictions. Furthermore, it can be seen that this condition state was assigned after several damage zones with severe cracking were detected. Visual inspection findings gave rise to an additional testing in 2013, which included chloride content measurements. The tests revealed high content of chlorides and significant carbonation depth, primarily at abutments and wings. This damage was repaired in 2014. Although test results showed that deterioration affected only certain parts of bridge elements (such as lower parts of abutments and wings), the repair was performed for the whole abutments, wings, girder and edge beams. The side view of the bridge after repair is shown in Figure 3.

**Figure 3. Side view of Weikendorf Bridge in 2016**

### 3.2. Testing campaign

The testing involved measurement of several material properties such as the concrete compressive strength, carbonation depth, pH-value, chloride content, and reinforcement depth. According to test documentation, core samples were primarily drilled in order to detect and document the length and width of cracks that were observed in abutments. The height at which these samples were taken varied between 0.5 m and 1.5 m. Additionally, the same samples were used to determine the compressive strength $f_c$ [N/mm$^2$] according to ONR 23303:2010 [22]. Test results are presented in Table 1, where core samples are marked with A and B to distinguish the side of abutment according to ground plan presented in Figure 2.

Concrete cover was measured only at abutments, where the measured horizontal reinforcement had the cover of circa 30 mm with a diameter $\phi$14 on a raster of 15 cm, while vertical reinforcement had the cover of circa 40 mm with a diameter $\phi$12 on a raster of 20 cm.

The chloride content, pH value, and carbonation depth were determined according to ÖNORM B 4706 [23], on abutments (AB), wings (W), girder (G) and edge beam (EB). Sketches of measured

**Table 1. Results of compressive strength $f_c$ [N/mm$^2$] measured on abutments**

| Core sample | Sample length [cm] | Max. crack depth [cm] | Max. crack width [mm] | Failure load [kN] | Concrete density [kg/m$^3$] | Compressive strength [N/mm$^2$] |
|---|---|---|---|---|---|---|
| A - 1 | 24 | 5.0 | 10 | 229.3 | 2320 | 33.4 |
| A - 2 | 36 | 18.0 | 10 | 237.4 | 2332 | 34.6 |
| A - 3 | 20 | 20.0 | 4 | 258.6 | 2347 | 37.7 |
| B - 1 | 39 | 18.0 | 11 | 325.3 | 2375 | 47.4 |
| B - 2 | 31 | 3.0 | 5 | 300.0 | 2332 | 43.7 |
| B - 3 | 24 | 4.5 | 5 | 271.8 | 2339 | 39.6 |

**Table 2. Chloride content, carbonation depth, and pH value results measured on bridge abutments (AB), wings (W), girder (G), and edge beam (EB)**

| Location (point) | Exposure class | Height [m] | Chloride content [wt.-%/c] | | | pH value [-] | | | Carbonation depth [mm] |
|---|---|---|---|---|---|---|---|---|---|
| | | | 0-15 [mm] | 15-30 [mm] | 30-45 [mm] | 0-15 [mm] | 15-30 [mm] | 30-45 [mm] | |
| AB 1 | XD3 | 0.20 | 2.20 | 2.20 | 1.20 | 9.33 | 10.73 | 12.17 | 38 |
| AB 2 | XD1 | 1.30 | 0.87 | 1.10 | 0.24 | 9.23 | 9.35 | 12.05 | 49 |
| AB 3 | XD1 | 2.35 | 0.22 | 0.28 | 0.20 | 9.26 | 11.99 | 12.60 | 52 |
| AB 4 | XD1 | 3.40 | 0.21 | 0.00 | 0.00 | 10.07 | 12.14 | 12.54 | 20 |
| AB 8 | XD3 | 0.20 | 3.90 | 2.30 | 2.50 | 9.23 | 9.23 | 9.23 | 18 |
| AB 7 | XD1 | 1.30 | 1.10 | 1.20 | 0.87 | 9.05 | 11.33 | 12.20 | 25 |
| AB 6 | XD1 | 2.35 | 0.37 | 0.41 | 0.18 | 9.65 | 9.65 | 9.65 | 46 |
| AB 5 | XD1 | 3.40 | 0.24 | 0.00 | 0.00 | 11.63 | 11.63 | 11.63 | 37 |
| W 1 | XD1 | 1.50 | 0.21 | 0.19 | 0.15 | 10.10 | 11.99 | 12.29 | 15 |
| W 2 | XD1 | 1.50 | 0.27 | 0.26 | 0.31 | 11.24 | 11.78 | 12.26 | 23 |
| W 3 | XD1 | 1.50 | 0.48 | 1.10 | 0.59 | 10.25 | 12.23 | 12.39 | 16 |
| W 4 | XD1 | 1.50 | 0.00 | 0.11 | 0.48 | 10.01 | 10.62 | 12.19 | 46 |
| G | XD1 | > 4.00 | <0.1 | <0.1 | <0.1 | 12.37 | 12.51 | 12.53 | 1 |
| EB | XD1 | > 4.00 | <0.1 | <0.1 | <0.1 | 12.44 | 12.45 | 12.57 | 1 |

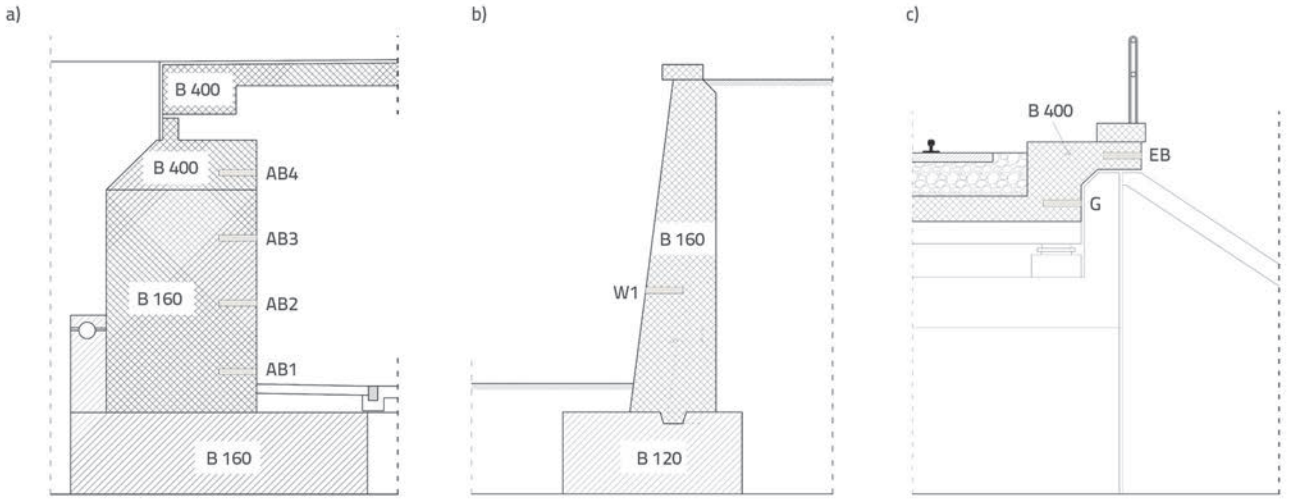


**Figure 4. Sketch of location of measured chloride content, pH values and carbonation depth in a) abutments, b) wings, c) girder and edge beam**

locations (points) in abutments, wings, girder, and edge beam, are shown in Figure 4. Concrete classes designed according to ÖNORM B 3302 [24] are also indicated in the sketches.

The chloride content, pH value, and carbonation depth results, are presented in Table 2. The chloride content and pH results value were obtained by analysing three concrete specimens at each location (each point). These specimens, measuring 15 mm in length, were taken at three different depths, namely 0-15 mm, 15-30 mm, and 30-45 mm. Points AB 1-4 are located on abutment B, and points AB 5-8 are located on abutment A, as shown on ground plan given in Figure 2.

### 3.3. Parameter identification

When considering the exposure classification given in the latest edition of EN 206-1 [11] for continental European countries without sea exposure, such as Austria, it should be noted that only the exposure classes XD are significant. The parameters considered dependent on exposure class are the substitute surface $\Delta x$, substitute surface chloride content $C_{S,\Delta x}$, and ageing exponent $\alpha$. The substitute surface is not considered in exposure classes XD1 and XD2 while, in exposure class XD3, it can be described by beta distribution with $\mu$ = 9 mm, $\sigma$ = 5, $a$ = 0, and $b$ = 50 [12]. Mean values of surface chloride content $C_{S,\Delta x}$ were determined as average values based on the chloride content $C(x,46)$ measured at the depths of 0-15 mm, as shown in Table 2. Points A1 and A8 were taken into account for exposure class XD3, and points A2 and A7 for exposure class XD1. These mean values $\mu$ of the surface chloride content $C_{S,\Delta x}$ are presented in Table 3. According to project DARTS [25], the chloride content on surfaces exposed to salt, other than seawater salt, is described with the Lognormal distribution and CoV = 0.75.

The ageing exponent $\alpha$ is also considered to be one of the parameters depending on exposure class. Moreover, it is believed to be dependent on cement type as well. Unfortunately, cement types used in the existing bridges are rarely documented. Hence, when performing the analysis of the limit state of depassivation, the cement type was considered as an unknown property. However, although cement types used in elements of Weikendorf Bridge are not known, it is possible to exclude certain types that were not produced in Austria at the time of construction.

**Table 3. Surface chloride content $C_{S,\Delta x}$ used in the analysis, based on measured chloride content at Weikendorf Bridge and project DARTS[25] recommendations**

| Exposure class | $C_{S,\Delta x}$ [wt.-/c] | |
|---|---|---|
| XD1 | $\mu$ = 0.99 | $\sigma$ = 0.74 |
| XD3 | $\mu$ = 3.05 | $\sigma$ = 2.29 |

Table 4 shows commonly used cement types in Austria based on the use of additives, with notations according to the current ÖNORM EN 197-1 [26] and the corresponding notations from previous standard ÖNORM B 3310 [27]. Indicative values of the chloride migration coefficient $D_{RCM,0}$ and ageing exponent $\alpha$ presented in *fib* Bulletin 76 [14] are also shown in Table 4. These values were collected through extensive research presented in Gehlen [28], DARTS [25], Bamforth [29], Lay [30], etc. The values of $D_{RCM,0}$ at early ages for most concretes seem to be equal to or lower, when compared with measured values of the apparent diffusion coefficient at a reference point in time $D_{app}(t_0)$ reported in the existing literature, [31, 32]. On the other hand, for the time periods exceeding 10 years, the values of $D_{app}(t)$ obtained by using RCM tend to be higher, which results in a conservative estimate of service life.

The chloride migration coefficient $D_{RCM,0}$ can be described via normal distribution, where mean $\mu$ is given in Table 4. The standard deviation $\sigma$ is equal to 0.2*$\mu$. The ageing exponent $\alpha$ is in exposure class XD1 considered independent of cement type and described with beta distribution with the mean value $\mu$ = 0.65, standard deviation $\sigma$ = 0.12, the lower limit a = 0.0, and the upper limit b = 1.0 [22]. On the other hand, the mean value

**Table 4. Cement types used in Austria (current and former notations) with the assigned indicative chloride migration coefficient $D_{RCM,0}$ and ageing exponent $\alpha$, based on *fib* Bulletin 76 [14], ÖNORM B 3310[27], and ÖNORM EN 197-1 [26]**

| 1963 - 1980 | 1980 - 1994 | 1994 | 1993 | $D_{RCM,0}$ (μ) [$*10^{-12}$ m²/s] | | | | | | α (μ / σ) [-] |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | | 0.35 | 0.40 | 0.45 | 0.50 | 0.55 | 0.60 | |
| - | PZ | PZ | CEM I | - | 8.9 | 10.0 | 15.8 | 19.7 | 25.0 | 0.30 / 0.12 |
| PZ (H) | PZ (H) | PZ (H) | CEM II/A-S | - | 7.0 | 8.0 | - | - | - | 0.35 / 0.16 |
| EPZ | EPZ | EPZ | CEM II/B-S | - | 5.0 | 7.7 | 8.3 | - | - | - |
| - | - | - | CEM II/A-D | - | 4.0 | 4.5 | 4.8 | 5.0 | - | 0.40 / 0.16 |
| - | - | - | CEM II/A-P | - | - | - | - | - | - | - |
| - | - | - | CEMII/B-P | - | - | - | - | - | - | - |
| - | - | - | CEMII/A-Q | - | - | - | - | - | - | - |
| - | - | - | CEMII/B-Q | - | - | - | - | - | - | - |
| PZ (F) | PZ (F) | PZ (F) | CEM II/A-V | 5.6 | 6.9 | 9.0 | 10.9 | 14.9 | - | 0.60 / 0.15 |
| | | FAZ | CEMII/B-V | - | - | - | - | - | - | 0.60 / 0.15 |
| PZ (F) | PZ (F) | PZ (F) | CEMII/A-W | - | - | - | - | - | - | - |
| - | - | FAZ | CEMII/B-W | - | - | - | - | - | - | - |
| - | - | - | CEMII/A-T | - | - | - | - | - | - | - |
| - | - | - | CEMII/B-T | - | 6.3 | 7.7 | 9.7 | - | - | - |
| PZ (T) | PZ (K) | PZ (K) | CEM II/A-L | - | - | - | - | - | - | - |
| - | - | - | CEM II/B-L | - | - | - | - | - | - | - |
| PZ (T) | PZ (K) | PZ (K) | CEM II/A-LL | - | 9.4 | 12.8 | 15.1 | - | - | 0.30 / 0.12 |
| - | - | - | CEM II/B-LL | - | - | - | - | - | - | - |
| PZ (C) | PZ (C) | PZ (C) | CEM II/A-M | - | - | - | - | - | - | - |
| - | - | CMZ | CEM II/B-M | - | - | - | - | - | - | - |
| - | - | - | CEM III/A | - | 3.9 | 3.9 | 4.2 | - | - | 0.40 / 0.18 |
| HOZ | HOZ | HOZ | CEM III/B | - | 1.4 | 1.9 | 2.8 | 3.0 | 3.4 | 0.45 / 0.20 |
| - | - | - | CEM III/C | - | - | - | - | - | - | - |
| - | - | - | CEM IV/A | - | - | - | - | - | - | - |
| - | - | - | CEM IV/B | - | - | - | - | - | - | - |
| - | - | - | CEM V/A | - | - | - | - | - | - | - |
| - | - | - | CEM V/B | - | - | - | - | - | - | - |
| - | - | - | CEM I | 5.6 | 6.9 | 9.0 | 10.9 | 14.9 | - | 0.60 / 0.15 |
| - | - | PZ HS | CEM I | 4.4 | 4.8 | - | - | 5.3 | - | 0.40 / 0.16 |

and standard deviation of ageing exponent in exposure classes XD2 and XD3 are considered to be cement dependent, as shown in Table 4.

The cement types highlighted in Table 4 might have been used for the construction of the Weikendorf Bridge, and were therefore used in the analysis. For other cement types that are not highlighted but were used at the time of construction, the analysis could not be performed due to the lack of data about the migration coefficient or ageing exponent.

Since the chloride migration coefficient $D_{RCM,0}$ has not been tested for some of the w/c ratios presented in Table 4 (e.g. w/c = 0.6, 0.7, and 0.8), the missing values were approximated using the linear fitting. Similar approximations of the influence of water-cement ratio on the diffusion coefficient can also be found in Vu and Stewart [33]. Just like the cement type, the w/c ratio is also rarely documented in design documents of existing bridges. When unknown from design documents, concrete properties such as cement type and w/c ratio can be determined by means of comprehensive petrographic analyses. However, in current bridge management practice, these properties are rarely measured in hardened concrete [34]. Nonetheless, the range of w/c ratio used for a particular bridge can be defined through information on concrete class and subsequent concrete strength. The connection of concrete classes and cement strength classes used at the time of construction is shown in Table 5. This connection was used to determine the approximate range of w/c ratio and its related parameter $D_{RCM,0}$.

**Table 5. Prescribed w/c ratios in connection to concrete classes and cement strength classes VÖZ [35]; with overall and average values assigned**

| Concrete class | w/c ratio per cement strength class | | | | | | Overall (average) |
|---|---|---|---|---|---|---|---|
| | Z 275 | | Z 375 | | Z 475 | | |
| B 160 | 0.74 | 0.92 | 0.87 | 1.03 | - | - | 0.74 - 1.03 (0.89) |
| B 225 | 0.61 | 0.75 | 0.71 | 0.83 | 0.77 | 0.88 | 0.61 - 0.88 (0.75) |
| B 300 | 0.51 | 0.64 | 0.61 | 0.71 | 0.66 | 0.74 | 0.51 - 0.74 (0.63) |
| B 400 | 0.32 | 0.54 | 0.51 | 0.61 | 0.57 | 0.64 | 0.32 - 0.64 (0.48) |
| B 500 | - | - | 0.39 | 0.52 | 0.48 | 0.56 | 0.39 - 0.56 (0.48) |
| B 600 | - | - | 0.32 | 0.43 | 0.37 | 0.48 | 0.32 - 0.48 (0.40) |

Concrete classes highlighted in Table 5 are the classes assigned in design documents as used in abutments and wings. Moreover, since the w/c ratios of concrete classes used in Table 5 are given in ranges, the w/c was considered as an unknown parameter.

As the value of initial chloride content $C_0$ can hardly be quantified without a chemical analysis, it was considered to be a constant value amounting to $C_0$ = 0.1 wt.-%/c. This value was chosen since the structure was built in 1960 and it was not possible to exclude potential chloride contamination originating from concrete constituents or from construction process.

The critical chloride content $C_{crit}$ is a parameter that is hard to quantify for a specific bridge since it depends on many factors such as the concrete-steel interface, chemical composition of steel, pH value of concrete pore solution, etc. According to ÖNORM B 4706 [23], and both the ÖVBB-Richtlinie [36] and ÖBV-Richtlinie [37], no inspections, monitoring or repair is needed for the chloride content lower than 0.6% if there is no corrosion. Furthermore, in *fib* Bulletin 34 [12] and *fib* Bulletin 76 [14] $C_{crit}$ is described by beta distribution with the mean value of μ = 0.6 wt.-%/c and standard deviation of σ = 0.15, where the lower boundary is $a$ = 0.2 and the upper boundary is $b$ = 2.0. However, it has to be noted that in some circumstances corrosion was not registered for values as high as 2-3 wt.-%/c [38].

As reported in previous subsection, the concrete cover was reported in the testing report of the bridge only as an approximate value measured in abutments. Moreover, the bridge blueprints show no indication of concrete cover on any elements. Therefore, since the concrete cover was reported in the testing report only as an approximate value in abutments, the mean value of concrete cover μ was considered as an unknown parameter, with the coefficient of variation of CoV = 0.20.

Based on the mean yearly temperature measured in the period from 1960 to 1990 in the State of Lower Austria [39], a mean value of μ = 281.6 K and standard deviation of σ = 1.2 K were adopted for ambient temperature $T_{real}$.

Overall material and environmental parameters applied for defining the limit state function $g(c,t)$ presented in Eq. (1) are shown in Table 6.

**Table 6. Overall parameters used for analysis of limit state function *g*(*c*,*t*), divided into known and unknown parameters**

| Known parameters | | Distribution | XD1 | XD3 | XD1 | XD3 | a | b |
|---|---|---|---|---|---|---|---|---|
| Parameter | Unit | | Mean μ | | Standard deviation σ | | | |
| $t_0$ | years | Constant | 0.0767 | | - | | - | - |
| $T_{ref}$ | K | Constant | 293 | | - | | - | - |
| $T_{real}$ | K | Normal | 281.6 | | 1.2 | | - | - |
| $b_e$ | K | Normal | 4800 | | 700 | | - | - |
| $C_0$ | wt.-%/c | Constant | 0.10 | | - | | - | - |
| $C_{crit}$ | wt.-%/c | Beta | 0.60 | | 0.15 | | 0.2 | 2.0 |
| $C_{S\Delta x}$ | wt.-%/c | Lognormal | 0.99 | 3.05 | μ * 0.75 | | - | - |
| $\Delta x$ | mm | Beta | 0 | 9 | 0 | 5 | 0 | 50 |
| **Unknown parameters** | | **Distribution** | **XD1** | **XD3** | **XD1** | **XD3** | **a** | **b** |
| **Parameter** | **Unit** | | **Mean μ** | | **Standard deviation σ** | | | |
| A | - | Beta | 0.65 | Tablica 3 | 0.12 | Table 3 | 0 | 1 |
| $C$ | mm | Normal | 20 – 50 | | μ * 0.20 | | - | - |
| $D_{RCM,0}$ | m²/s | Normal | Tablica 3 | | μ * 0.20 | | - | - |

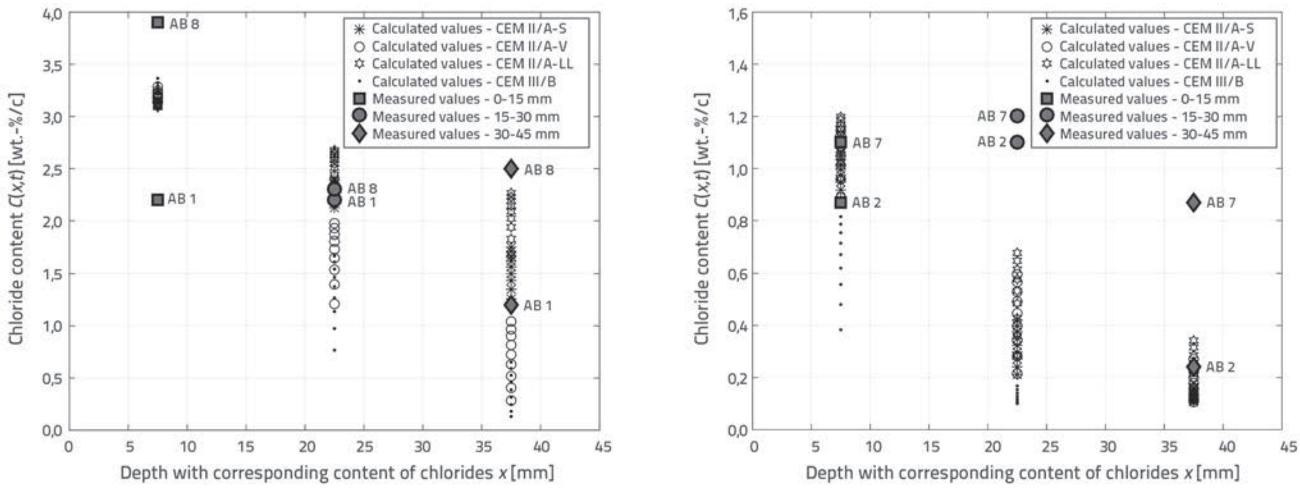


**Figure 5. Comparison of measured and calculated chloride profiles at abutments for the height of a) 0.20 m, and b) 1.30 m**

## 3.4. Comparison of measured and calculated chloride profiles

A statistical elaboration of measured values can be performed in a perfect case, i.e. when substantial information on chloride content $C(x,t)$ obtained by testing campaign is available. Such elaboration would result in determining properties of distribution for each tested element, as well as for each specific bridge exposure region. In such cases, it would be possible to use measurement results to update the previously calculated chloride content, e.g. by means of Bayesian updating. However, in the case of the Weikendorf Bridge, only two measurements were available for locations (points) with same heights and distance to the road, which did not allow statistical analysis of measured values. Hence, the measured chloride contents $C(x,t)$ presented in Table 2 were used only to approximate some parameters of the chloride ingress model presented in Eq. (2), such as $C_{S,\Delta x}$.

A comparison of measured and deterministically calculated chloride profiles was performed in order to better comprehend the results obtained by testing, and to validate overall parameters chosen for the analysis (presented in Table 6). Measured values were obtained by analysing three concrete specimens measuring 15 mm in length, at three different depths, i.e. 0-15 mm, 15-30 mm, and 30-45 mm. These values were compared with calculated values at average depths of tested samples, namely at 7.5 mm, 22.5 mm, and 37.5 mm. The results of both measured and calculated chloride content for the abutment at the heights of 0.2 m and 1.30 m are presented in Figure 5.

The analysis of calculated values was performed for various cement types used in Austria at the time of construction, as well as for various w/c ratios. The input parameters used to obtain calculated values were adopted as presented in Table 6, considering the height of 0.20 m being in XD3 exposure class and the height of 1.30 m being in XD1 exposure class. When compared to Table 6, the only difference is that higher chloride content at substitute surface $C_{S,\Delta x}$, namely $C_{S,\Delta x}$ = 1.5 wt.-%/c was taken into account at the height of 1.30 m. Such value is the maximum suggested by the *fib* Bulletin 34 [12] and *fib* Bulletin 76 [14] for XD1 exposure class, and it was taken into account since points at 1.30 m are still relatively close to XD3 class, which exhibits much higher $C_{S,\Delta x}$ values.

In the same manner, Figure 6 depicts the comparison for bridge wings, in which only one measurement was made per each wing, at the height of 1.5 m. The $C_{S,\Delta x}$ value corresponding to the one used in Table 6 was considered in the calculations performed for the wings.

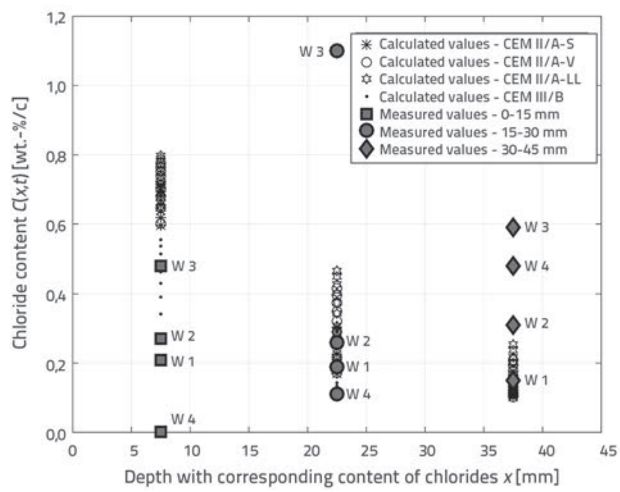


**Figure 6. Comparison of measured and calculated chloride profiles for wings at height 1.5 m**

## 3.5. Analysis of limit state of depassivation

The serviceability limit state of chloride-induced depassivation is assumed to be reached when the chloride content $C(c,t)$ at reinforcement depth exceeds the critical chloride content $C_{crit}$. Both quantities are of uncertain nature, i.e. they are introduced as random quantities and contrasted in the random limit state function $g(c,t)$, as presented in Eq. (1). The probability of the limit state function $g(c,t)$ being less than zero is regarded as failure probability $p_f(t)$, as shown in Eq. (5).

$$p_f(t)=p_{dep}=p\{g<0\}=p\{c-xc(t)\} \qquad (5)$$

The reliability (safety) index β demonstrates how often the standard deviation $s_g$ of the limit state function $g(c,t)$ may be placed between zero and the mean value $m_g$.

$$\beta(t) \approx \frac{m_g(t)}{s_g(t)} \qquad (6)$$

The moment when the probability of failure $p_f(t)$ (or reliability index $\beta(t)$) exceeds target values is considered as the end of service life, and the time from construction until that moment is considered as duration of service life $t_{SL}$.
The probabilistic engine FReET (*Feasible Reliability Engineering Tool*) was used for calculating the probabilities of reaching the limit state and subsequent reliability indices β, [40]. Computed values of reliability indices β, for exposure classes XD1 and XD3 to which elements of Weikendorf Bridge belong, are presented in Figure 7 and Figure 8. These figures show the development of the reliability index with variation of parameters considered as unknown (w/c ratio, concrete cover, and cement type) for the service life $t_{SL}$ = 100 years.
In order to comprehend the change of probability of failure over time $p_f(t)$, the reliability spectrum for the most favourable and the most unfavourable cement type (CEM III/B and CEM II/A-LL, respectively) used in Austria at the time of construction of the Weikendorf Bridge is shown in Figure 9. The probabilities were calculated for w/c ratios 0.4, 0.5 and 0.6, in the exposure classes XD1 and XD3, for a concrete cover of 35 mm.

## 3.6. Target reliabilities for limit state of depassivation

In order to use the model and to determine the duration of service life, and thus to schedule an optimum

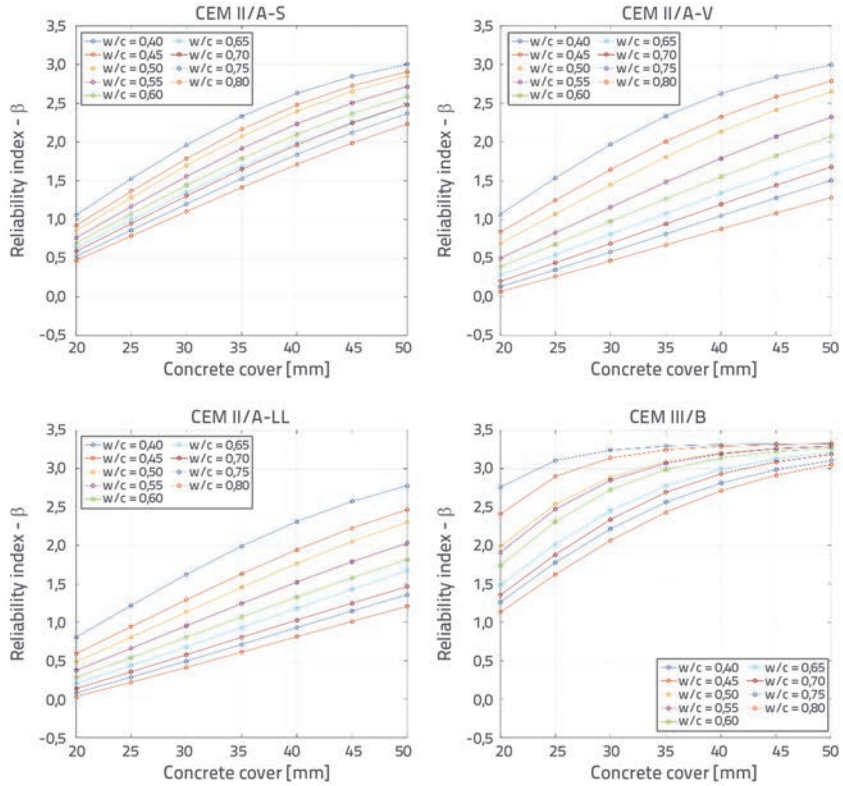


**Figure 7. Indices of reliability β for the age $t_{SL}$ = 100 years, presented in the combination of different w/c ratios and depths of reinforcement, in exposure class XD1**

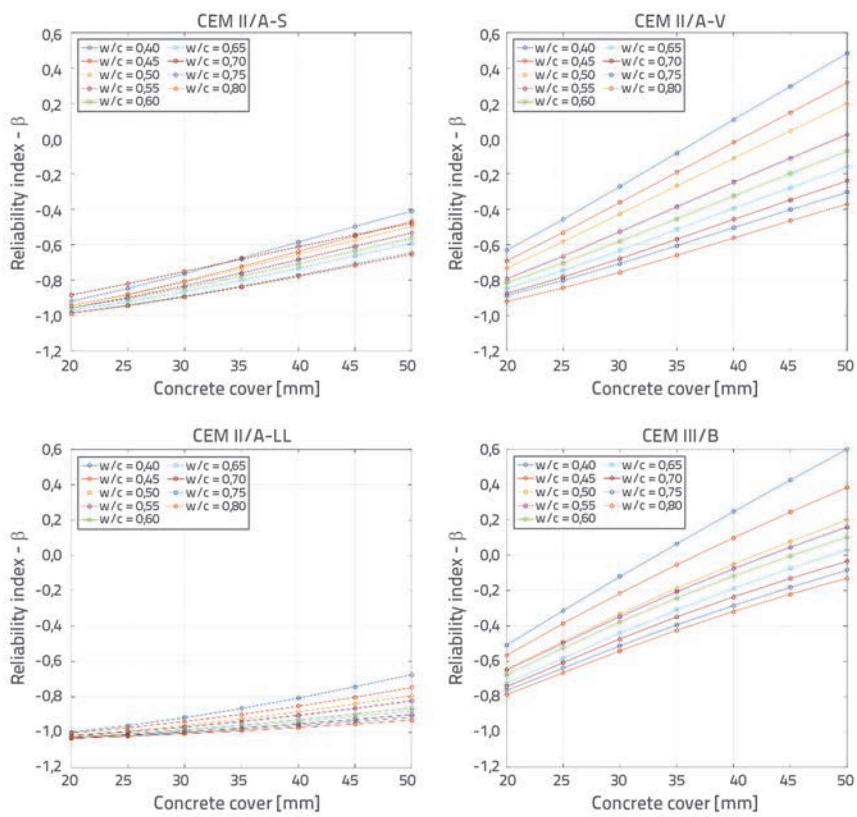


**Figure 8. Indices of reliability β for the age $t_{SL}$ = 100 years, presented in the combination of different w/c ratios and depths of reinforcement, in exposure class XD3**

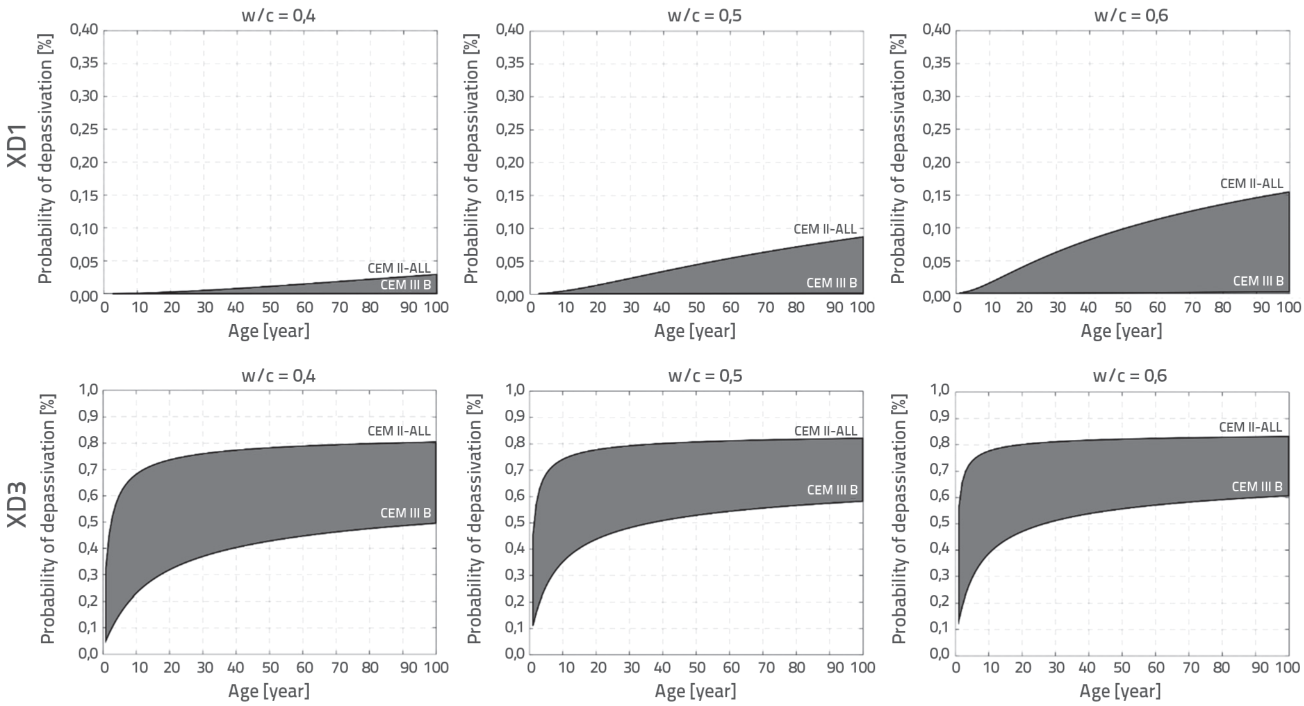


**Figure 9. Spectrum of probability of failure $p_f(t)$ for the favourable (CEM III/B) and unfavourable cement type (CEM II/A-LL) in exposure classes XD1 and XD3 for w/c ratios 0.4, 0.5, and 0.6**

maintenance strategy, a verification should be performed by comparing the obtained reliability with the target reliability.

The *fib* Bulletin 34 [12] distinguishes reliability classes (RC1, RC2 and RC3) with respect to the consequence of failure. For the limit state of depassivation caused by de-icing salts, in all the different RC classes for a reference period of 50 years, an equal design targeted reliability is proposed, i.e. $\beta$ = 1.3 ($p_f \approx$ 0.09).

Andrade [38] proposes targeted depassivation probabilities (for prediction of service life) in relation to consequences of the corrosion as "low, moderate and high", with the value of 50% for low and moderate consequences and 25% for high consequences. Von Greve-Dierfeld and Gehlen [41] divide target reliabilities according to exposure classes, thus distinguishing the situations where high corrosion rates are expected after depassivation due to presence of water and oxygen.

### 3.7. Sensitivity analysis

Since it is not feasible to precisely test or measure a dozen of parameters of the model, at least the most influential parameters should be known to use the model in real situations. Hence, the significance of a particular parameter for the limit state of depassivation caused by ingress of chlorides was examined for the case of the Weikendorf Bridge. An approach that uses nonparametric rank-order statistical correlation between the basic random variables and the structural response variable is implemented using the FReET software. Since structural response models are generally nonlinear, a non-parametric rank-order correlation is used by means of the Spearman or Kendall rank correlation coefficient [42].

The sensitivity factor $\alpha_i$ was analysed for different combinations of exposure classes (XD1 and XD3) and cement types (CEM II/A-LL and CEM III/B), presuming the w/c = 0.5. The corresponding results are presented in Figure 10. It should be noted that in Figure 10 all the sensitivity factors are given as absolute values in order to be mutually comparable. In reality, sensitivity factors of parameters $C_{S,\Delta x}$, $D_{RCM,0}$, $\Delta x$ and $T_{real}$ are negative, meaning that the limit state function $g(c,t)$ decreases with their increase.

## 4. Discussion of results

As can be seen in Figure 5 and Figure 6, there is a great nonconformity between the chloride contents measured at identical heights and depths, as well as between the measured and calculated contents. In addition, the decrease of the measured chloride content with the increase of depth is irregular, as can easily be seen in Figure 6. The difference between the values measured at the same locations in different abutments and wings can be explained by possible influence of cracks and specific microenvironment in each location. Furthermore, the nonconformity of measured and calculated values could be caused by some of the previously described shortcomings of the model. However, the information on measured results was too scarce to grasp to what extent some of these shortcomings could shape the difference between the measured and calculated values.

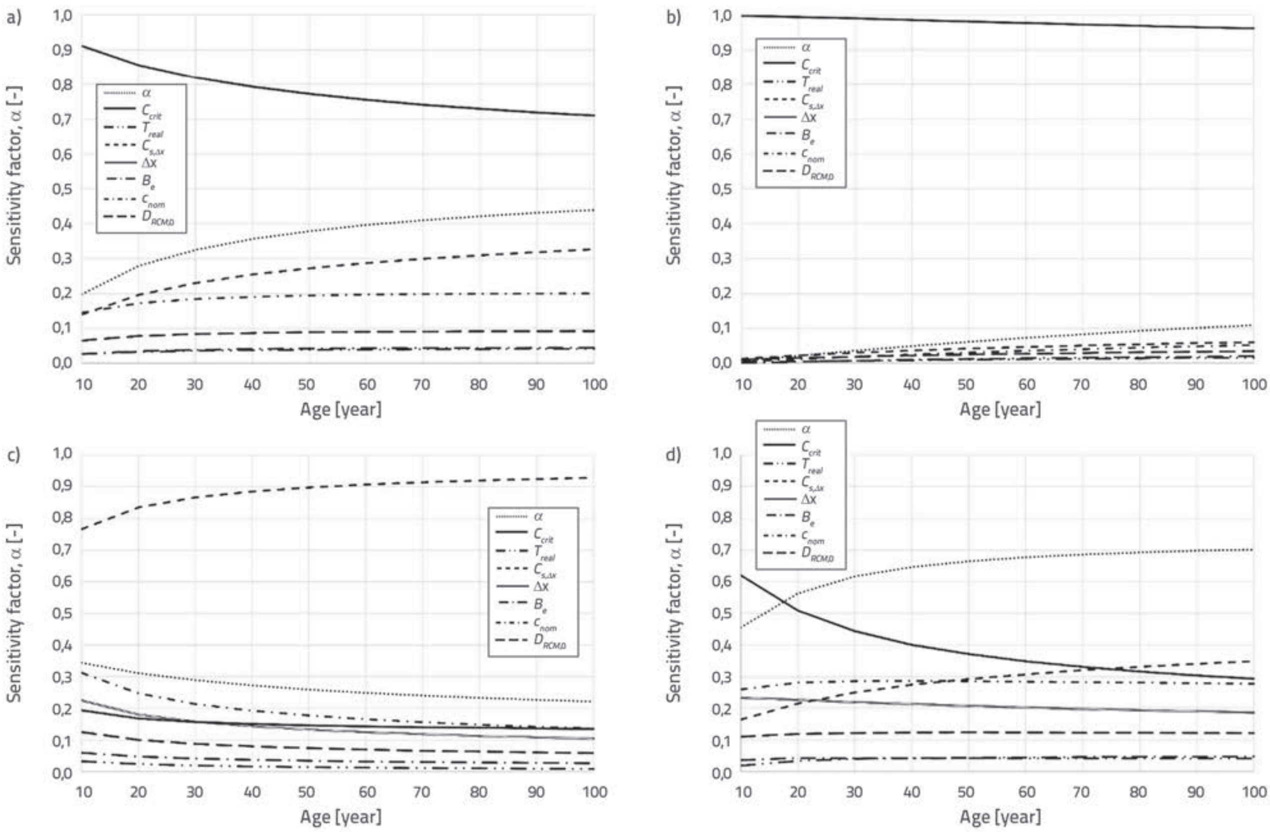


**Figure 10. Sensitivity factor $\alpha_i$ for parameters of chloride ingress model in dependence on age of Weikendorf Bridge *t*, for different parameter combinations: a) XD1 & CEM II/A-LL, b) XD1 & CEM III/B, c) XD3 & CEM II/A-LL, and d) XD3 & CEM III/B**

The chloride content at the substitute surface $C_{S,\Delta x}$ in exposure class XD1, as used for the analysis of the limit state of depassivation, was based on points A2 and A7 only, which have the highest chloride content compared to all other points in exposure class XD1. However, other points measured at different heights in abutments, as well as the points measured in wings, show lesser values, and thus, in reality, a lower $C_{S,\Delta x}$ should be taken into account for these locations. Although the highest chloride content is taken into account, the chloride ingress in class XD1 exhibits low probabilities of depassivation.

According to design documents, concrete class B160 was used in abutments and wings. Such concrete class would mean that the w/c ratio ranging from 0.71 to 1.03 was used. However, by screening the ÖBB database, it can be seen that the concrete class B300 is usually used in bridge abutments and wings, and that B225 is the lowest class used. It was therefore assumed that a concrete class higher than the one assigned in design documents was actually used. For this reason, the highest w/c ratio that was used in the analysis of the limit state of depassivation amounted to 0.80.

Assuming the reliability index $\beta = 0$ is the target reliability, it can be seen in Figure 7 that this target reliability will never be overpassed in exposure class XD1 in the period of 100 years. On the other hand, when considering exposure class XD3 depicted in Figure 8, it can be seen that parameter combinations exhibiting reliability index higher than 0 occur in a very limited number of cases. The stated difference in reliability emphasises the strong influence of parameters $C_{S,\Delta x}$, $\alpha$ and $\Delta x$, which significantly differ in these classes. In addition, a low reliability index can be partly explained by the fact that the approach based on chloride migration coefficients $D_{RCM,0}$ results in higher diffusion coefficients $D_{app}(t)$ when compared to the approach based on diffusion tests performed on real structures.

Although the topic of exact duration of design service life in bridges has attracted significant attention in research community, it can be seen in Figure 9 that a minor increase in probability of depassivation $p_f(t)$ occurs after the 50th year. When different w/c ratios are observed, the probability of depassivation $p_f(t)$ increases in classes XD1 and XD3 by max. 0.07 in the period between the 50th and the 100$^{th}$ year.

The aim of the sensitivity analysis was to identify crucial parameters of the model with respect to full probabilistic analysis. It can easily be seen in Figure 10 that $C_{crit}$ presents itself as the parameter which influences the limit state function $g(c,t)$ the most, especially in cases when there is a low probability of depassivation. However, in class XD3, where the depassivation is more likely to happen, parameters such as $C_{S,\Delta x}$ and $\alpha$, exert a stronger influence on the limit state function $g(c,t)$.

The main objective of the analysis performed in the case study was to investigate applicability of the chloride ingress model in real situations, where some of the parameters are not fully known. For this reason, the influence of parameters that are measurable, but are unknown, was studied. If the failure probability spectrum shown in Figure 9 is observed, it can be seen that two different cement types (favourable and unfavourable) exhibit very large difference in calculated probabilities of failure.

Other two parameters denoted as unknown were the concrete cover *c* and w/c ratio. The influence of both parameters on the reliability index $\beta$ can be seen in Figure 7 and Figure 8, where significantly lower influence can be seen in exposure class XD3. The maximum error in determining $\beta$ when assigning w/c ratio is less than 0.2 in class XD3 and 0.5 in class XD1, for a difference of $\Delta$(w/c) = 0.05. The same maximum error is observed for the difference of concrete cover $\Delta(c)$ = 5 mm. In other words, if a value of 35 mm instead of 30 mm is assigned to a concrete cover while performing an analysis, the maximum error in determining the reliability index $\beta$ is 0.2 for class XD3 and 0.5 for class XD1.

In general, it can be seen that high probabilities of depassivation were obtained in the analysis, especially for elements belonging to exposure class XD3, as shown in Figure 9. These are the most visible for cement type CEM II/A-LL, which exhibits a 70% chance of depassivation during the first ten years, regardless of w/c ratio. If these probabilities are observed for the age of 46 years, at which it was documented that Weikendorf Bridge was already significantly affected by cracking, the results of the analysis show a correspondence with the real situation. However, one has to bear in mind that, while performing probabilistic analysis of limit state with multiple parameters, statistical values of some of these parameters incorporate the uncertainty in knowledge on parameters, rather than real inherited uncertainty of these parameters. In order words, by not having the exact information on parameters such as $C_{S,\Delta x}$, $\Delta x$, $T_{real}$, etc., high standard deviations (which are found in literature) are usually assigned to these parameters. However, these parameters may not vary as much as assigned in the bridge under study. The accumulation of high standard deviations of multiple variables results in high standard deviation of limit state function $g(c,t)$ and thus in a low reliability index $\beta$ and high probability of failure $p_f$. Such probability of failure represents the probability at which one can not exclude the failure regarding the current knowledge on structure and its properties, rather than actual probability that the structure will fail. In that manner, it can easily happen that many of existing bridges show very high probabilities of depassivation when analysed, but exhibit low chloride content when tested.

## 5. Conclusions

A study on the use of the chloride ingress model supported by *fib* for preliminary condition assessment in bridge management was presented in the paper.

In the current practice, preliminary assessment of existing bridges performed by most management agencies is based on visual inspection, rather than on the knowledge about material and structural properties, and environmental load. Because of that, and because of recent enhancements to the knowledge pool on the chloride ingress model, possible implementation of the model for preliminary assessment, as a supplement to current practice, is also studied in the paper.

The model was applied via a case study of the Weikendorf Bridge, which is characterized, just like most existing bridges, by the lack of material and load-related data. In the study, parameter identification was performed using bridge characteristics gathered from design and inspection documents, based on which the analysis of the limit state of depassivation was conducted. Along with the limit state analysis, target reliabilities were discussed, as well as the sensitivity and influence of crucial parameters of the model.

Based on the study of the Weikendorf Bridge, the following conclusions can be drawn:

- the analysis showed that regardless of the combination of unknown parameters, there is a high probability that elements in exposure class XD1 will not become depassivated within the period of 100 years. On the other hand, there is a high probability that elements belonging to class XD3 will become depassivated early in their service life, regardless of the quality of concrete used;
- if an element has not become depassivated by the age of 50 years, there is a low probability that it will become depassivated later. This indicates that the value of additionally obtained information on parameters of the model is low for bridges older than 50 years;
- the analysis of most favourable and unfavourable cement types possibly used at the time of construction showed a very high variation of results. Regardless of which probability of depassivation is considered as target, at least twice longer duration of service life is obtained when favourable cement is used. This leads to a conclusion that it is not feasible to utilize the model in its current form in cases where the exact cement type is not documented;
- concrete cover and w/c ratio have been recognized as key performance indicators for the time of depassivation and progression of reinforcement corrosion (see [43] and [44]). However, when the chloride ingress formulation shown in Eqs. (1) to (4) is analysed, these parameters show less influence on the limit state of depassivation, especially in case of exposure class XD3. It can therefore be concluded that the model can be used for preliminary assessment in cases when only approximate values of these parameters are known;
- since a high number of material parameters can not be determined for the bridge under study, nor for numerous bridges in infrastructure networks, documents such as "birth certificates" ought to be strongly substantiated. These documents should provide most relevant durability related

information crucial to engineers performing the assessment. Based on the performed study, in the case of chloride ingress, such information should comprise the cement type, exposure class, w/c ratio, and concrete cover. Besides these properties, management agencies should keep record on the use of de-icing salts in winter, in order to have at least approximate indication on chloride content at the outer concrete surface.

Based on the research presented in this paper, it can generally be concluded that deterioration models, such as the chloride ingress model, should become widely used in bridge management in the future, once the durability related information becomes regularly documented for all bridges. In combination with simple visual inspections, these models should give the indication on when bridges ought to be subjected to maintenance interventions or to more thorough inspections, testing and monitoring activities. However, in the current situation, when crucial parameters are not documented and are expensive to test, the utility of the model for predicting duration of service life, i.e. for scheduling future maintenance activities, is still rather limited.

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