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# Prediction of the remaining service life of existing concrete bridges in infrastructural networks based on carbonation and chloride ingress

Ivan Zambon[*1], Anja Vidović[1], Alfred Strauss[1], Jose Matos[2] and Norbert Friedl[3]

[1]*Department of Civil Engineering and Natural Hazards, University of Natural Resources and Life Sciences, Peter-Jordan-Straße 82, 1190 Vienna, Austria*
[2]*Civil Engineering Department, Campus de Azurém, Minho University, Guimarães, Portugal*
[3]*Bridge Construction and Structural Engineering, ÖBB-Infrastruktur AG, Nordbahnstraße 50, 1020 Vienna, Austria*

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**Abstract.** The second half of the 20th century was marked with a significant raise in amount of railway bridges in Austria made of reinforced concrete. Today, many of these bridges are slowly approaching the end of their envisaged service life. Current methodology of assessment and evaluation of structural condition is based on visual inspections, which, due to its subjectivity, can lead to delayed interventions, irreparable damages and additional costs. Thus, to support engineers in the process of structural evaluation and prediction of the remaining service life, the Austrian Federal Railways (Ö BB) commissioned the formation of a concept for an anticipatory life cycle management of engineering structures. The part concerning concrete bridges consisted of forming a bridge management system (BMS) in a form of a web-based analysis tool, known as the LeCIE_tool. Contrary to most BMSs, where prediction of a condition is based on Markovian models, in the LeCIE_tool, the time-dependent deterioration mechanisms of chloride- and carbonation-induced corrosion are used as the most common deterioration processes in transportation infrastructure. Hence, the main aim of this article is to describe the background of the introduced tool, with a discussion on exposure classes and crucial parameters of chloride ingress and carbonation models. Moreover, the article presents a verification of the generated analysis tool through service life prediction on a dozen of bridges of the Austrian railway network, as well as a case study with a more detailed description and implementation of the concept applied.

**Keywords:** service life; bridge management system; infrastructure management; concrete bridges; deterioration; concrete carbonation; chloride ingress

## 1. Introduction

In transportation infrastructural networks, bridges and viaducts are commonly recognized as the elements with highest significance and yet of very high vulnerability (Liu and Frangopol 2006, Marzouk *et al.* 2014). The performance of bridges deteriorates over time as a combined effect of mechanical stressors, harsh environment, and extreme events (Hakim and Abdul Razak 2014, Frangopol and Soliman 2016). Furthermore, when dealing with the service life of existing bridges, infrastructure operators often face many other problems, such as design and construction with a limited set of rules, varying quality of material and workmanship, and inadequate maintenance (Njardardottir *et al.* 2005). The extension of service life is considered to be essential in order to reduce life cycle costs and generally to improve sustainability (Strauss *et al.* 2012, Bocchini *et al.* 2013, Abé 2015).

Enhancement of concrete technology in the 20th century, as well as the boom in building and reconstruction of infrastructure after the Second World War (Pellegrino *et al.* 2011), led to a significant amount of bridges being made of reinforced concrete. Although concrete is a durable material that performs satisfactory during the envisaged service life, still, the beginning of the new century brought pending problems with infrastructure maintenance as infrastructure aged worldwide (Feng 2009, Adhikari *et al.* 2014, Boller *et al*. 2015). This trend can easily be seen from the reports of US Federal Highway Administration (FHWA), where in the early 1990s around one-third of bridges in the American main road network was already in the state of severe damage (Zanini *et al.* 2017). Moreover, in Europe, according to the data collected from 17 European railway administrations, almost 67% of railway bridges was older than 50 years, of which 36% was older than 100 years (PANTURA, 2011).

Gattulli and Chiaramonte (2005) defined a bridge management system (BMS) as any system or collective series of engineering and management functions comprising the actions necessary to manage a bridge network. A fundamental function of a BMS is to assist infrastructure operators in determining the best maintenance, repair and rehabilitation strategy with respect to current and future performance (Hasan *et al.* 2015). Throughout the years, wide varieties of approaches have been developed in managing bridges, most of them being computerized. In Europe only, numerous projects focused on the

∗Corresponding author, Ph.D. Student
E-mail: ivan.zambon@boku.ac.at


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infrastructure management and its standardization, as well as the assessment of concrete structures, their life cycle management and service life identification. Some of these projects are BRIME (2001), COST 345 (2004), Rehabcon (Fagerlund 2004), SAMARIS (2005), SAMCO (Rücker *et al.* 2006), CONREPNET (Matthews 2007), INNOTRACK (Ekberg and Paulsson 2010), ETSI Project (Salokangas 2013), COST TU1406 (2017), etc.

In 2008, The International Association for Bridge Management and Safety (IABMAS) prepared the questionnaire for collecting data on BMSs around the world (Tanasic 2016). In the last report from 2014, which analysed 25 different BMSs, it was recorded that nineteen of the systems can predict deterioration and subsequent duration of service life, where twelve of these systems use probabilistic methods (Mirzaei *et al.* 2014). By screening attached questionnaires, one can easily notice that the most used prediction models are the statistical models based on Markov property. These models use condition ratings from visual inspections as input (Firouzi and Rahai 2013). Among different types of models which use Markov property, the simple homogeneous Markov chain model is predominantly implemented in BMSs up until now. For the comprehensive comparison of different statistical models based on data from visual inspections, see Zambon *et al*. (2017).

However, to date, no formalized national approach has been accepted for infrastructure management in Austria. Therefore, the Austrian Federal Railways (ÖBB) initiated the formation of a comprehensive approach for lifecycle management of engineering structures. Based on that, a web-based bridge management system (BMS) named the LeCIE_tool is being developed for managing concrete bridges in railway network; its background will be described in this article. The distinctiveness of the tool is the implementation of physical models of deterioration based on the recommendations given in Bulletins 34 (2006), 59 (2011) and 76 (2015) of the International Federation for Structural Concrete (*fib*) as fundamental means of service life prediction.

Physical models, contrary to statistical models, take into account the damage-causing processes (Puz and Radic 2011) and do not depend on the subjectivity of visual inspection. In other words, they describe deterioration based on the environmental loads and relevant material parameters. Thus, they do not consider deterioration as a complete process, rather they just consider particular phenomena causing deterioration. So far, several researchers discussed the possibility of implementing physical models into service life prediction and bridge management; such are for example Maekawa *et al.* (2003), Hallberg and Racutanu (2007), Papadakis *et al.* (2007), Shin *et al.* (2011), and Ghodoosi *et al.* (2014). However, as far as authors' knowledge goes, physical models in the form given in *fib* Bulletins 34 (2006), 59 (2011), and 76 (2015) have not yet been implemented in any of the operational BMSs. One reason more for their implementation is that recently, significant research in the area of durability and benchmarking of properties of concrete has been made, which enables a more accurate application of physical models in service life prediction.

Chloride- and carbonation-induced corrosion models are chosen since these can clearly be identified as main causes of deterioration in transportation infrastructure (*fib* Bulletin 59, 2011, Papadakis 2013) and broadly accepted models exist for these processes.

## 2. Austrian Federal Railways' bridge stock

As of January 2016, the Austrian Federal Railways (ÖBB) has an infrastructure network of 5,406 railway bridges with a total length of approximately 90 km. For administrative reasons, only structures with a span larger than 2.00 m are formally referred to as railway bridges, and structures with a smaller span as culvert bridges. Culvert bridges make the second-largest portion of ÖBB's bridge stock with 2,781 items. In addition, there are 643 road bridges, 68 pedestrian bridges, and 210 other bridges, totalling 9,108 bridges with 13,416 girders owned by ÖBB. Fig. 1 presents the overall bridge stock with the number of bridges and girders.

These structures are predominantly made of reinforced concrete, which is mainly used for spans of up to 20 m. Bridges of a larger span were previously erected either in steel or as a stone/brick arches, and only more recently with reinforced concrete or composite structures. Fig. 2 depicts bridge distribution by material in the ÖBB bridge stock.

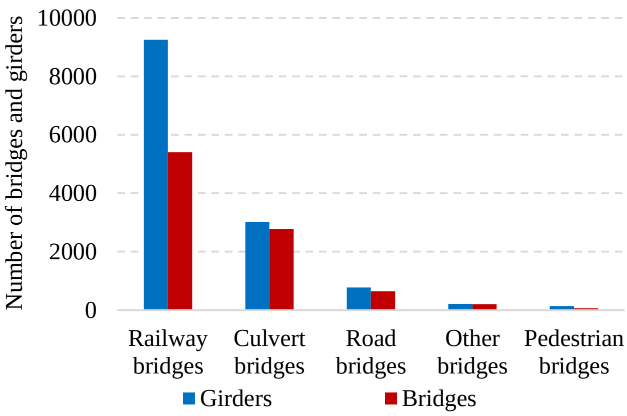



Fig. 1 Bridge stock of the ÖBB in 2016, arranged according to their general function

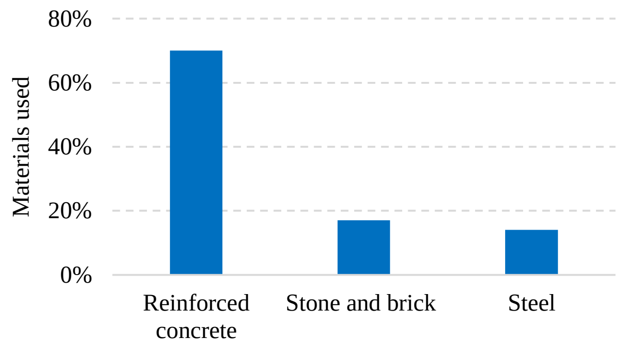



Fig. 2 Bridge distribution by material in the ÖBB bridge stock in 2016

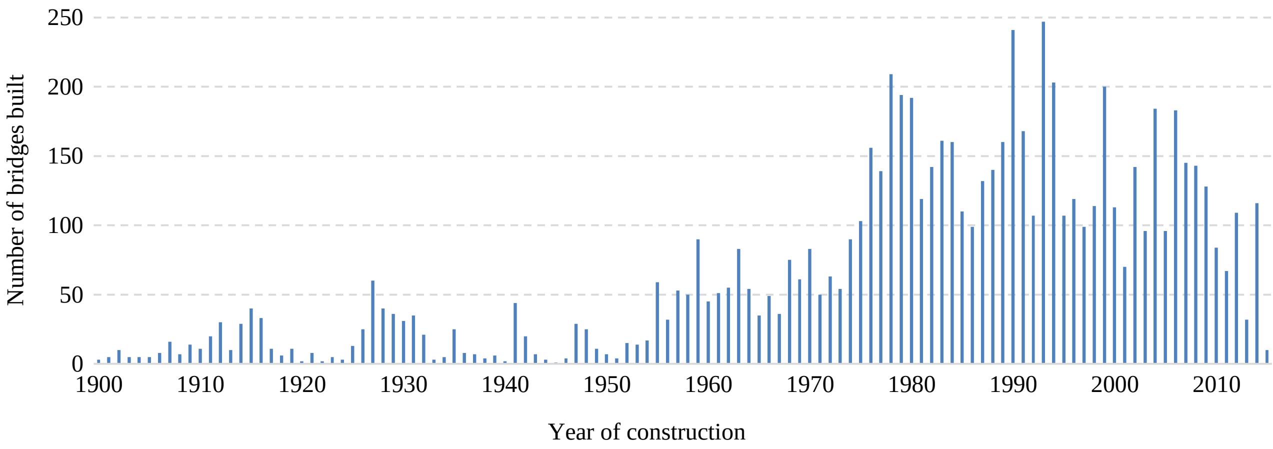



Fig. 3 Years of construction of reinforced concrete bridges in the ÖBB bridge stock in 2016

Table 1 Condition state grading system with classes used by the ÖBB

| | |
|---|---|
| Class 1 | Very good condition, no restrictions |
| Class 2 | Good state of preservation, no restrictions |
| Class 3 | Poor state of preservation, no restrictions |
| Class 4 | Very poor state of preservation, no restrictions |
| Class 5 | Very poor state of preservation, immediate restrictions |

Concrete and reinforced concrete bridges have been erected from around 1900 onwards, and especially in high frequency from 1945 onwards, as shown in Fig. 3. It is to be expected that reinforced concrete bridges will have the largest share in the next decades as well.

The inspection of bridges in ÖBB is carried out according to "Instandhaltungsplan Konstruktiver Ingenieurbau" [Maintenance Plan of Engineering Structures] regulations (OEBB Infrastruktur Regelwerk 06.01.02, 2012). In the case of structures built before 1980, these inspections take place every two years, while in the case of more recent structures , examination and assessment take place every three years.

In the course of assessment, the inspector assesses the damages found, determines the measures to be implemented and causes to be taken into account, assigns the condition rating and documents it in a report. The condition state is based on a grading system with classes 1 to 5, which refers to all components and safety risks of the installation, as presented in Table 1.

Bridges in Classes 3 and 4 are subject to repairs, while bridges in the case of Class 5 (and in some cases Class 4) are subjected to renewal.

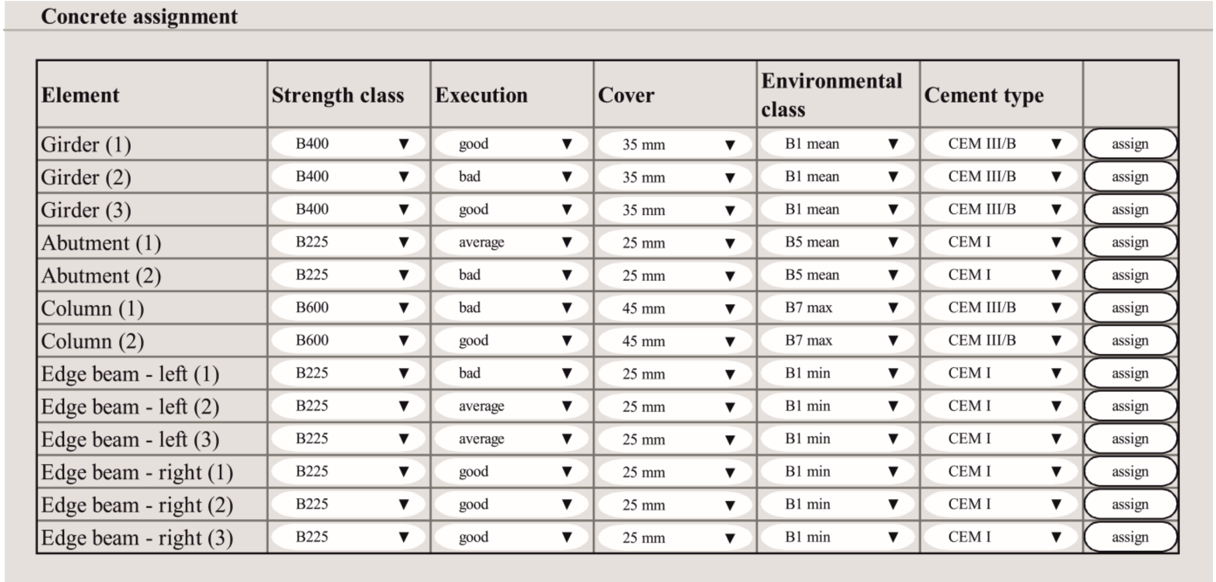
## 3. LeCIE_tool

LeCIE_tool is a BMS in the form of a web-based analysis tool that is currently being developed at the University of Natural Resources and Life Sciences, in Vienna, Austria. It is a part of a larger project funded by the Austrian Federal Railways (ÖBB) for the development of a comprehensive concept for an anticipatory life cycle management of engineering structures. The tool is in its early stages of development and it is, at this moment, primarily focused on concrete structures.

User interface of the tool consists of several parts, main being the tabs for administration, routes, maps and literature. The administration tab comprises the sub-tabs for the input of new bridges, and tabs for editing bridges or route properties already listed. Routes tab shows the list of routes with assigned bridges, and the maps show a Google-Earth map position of listed bridges. The literature tab provides the important literature used during the creation of the tool and tutorials for users.

When performing an input of a new bridge, the user has to assign the main information regarding the bridge, such as the name, route, object number, dimensions, etc. In addition, the user has to model the bridge with defining bridge elements and their material properties. Entering material properties, and thus deterioration characteristics, is divided in two modes, the first mode being the default mode and the second one being the detailed mode. In the default mode, which is depicted in Fig. 4, the user can assign only several main material parameters, and based on these the deterioration characteristics implemented as default are being automatically assigned.

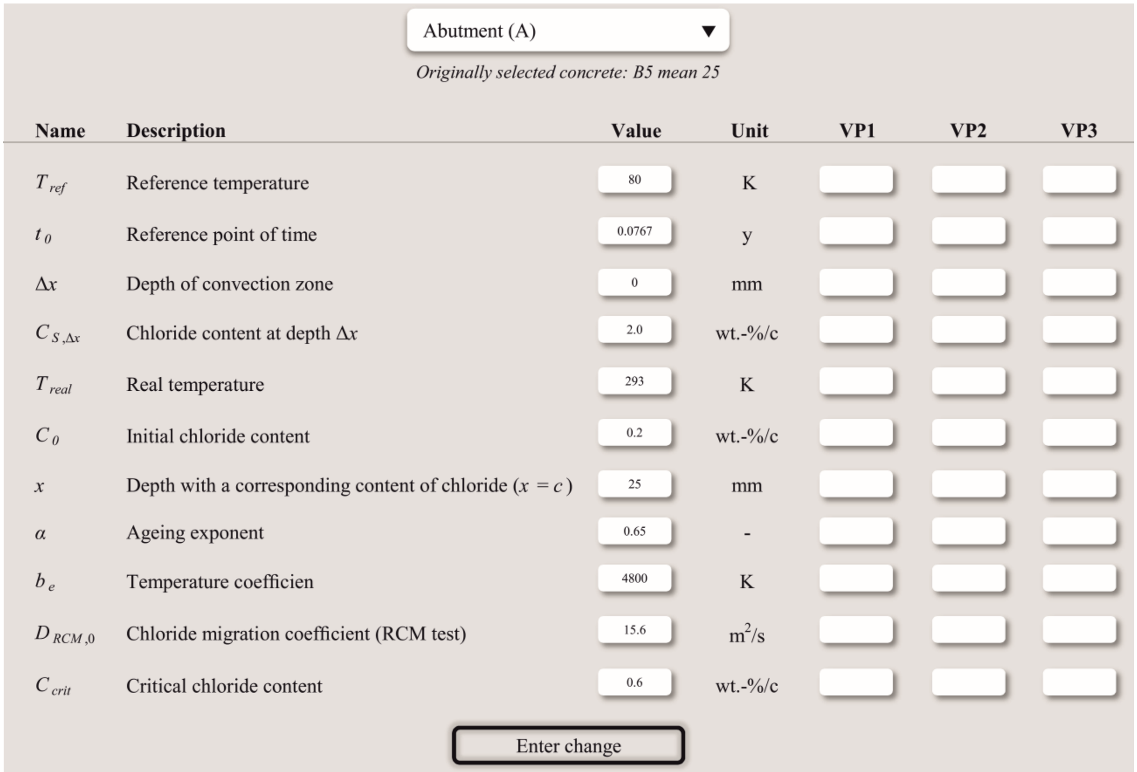
From the drop-down lists in the default mode the user can assign concrete strength classes according to standard ÖNORM B 4710 (2004), execution quality (poor, average or good) and concrete cover (25 mm, 35 mm or 45 mm). Moreover, the environmental concrete classes (explained in following chapters) with associated environmental loading and cement types can also be assigned. In this way, when no detailed information is available, an engineer entering



**Concrete assignment**

| Element | Strength class | Execution | Cover | Environmental class | Cement type | |
|---|---|---|---|---|---|---|
| Girder (1) | B400 ▼ | good ▼ | 35 mm ▼ | B1 mean ▼ | CEM III/B ▼ | assign |
| Girder (2) | B400 ▼ | bad ▼ | 35 mm ▼ | B1 mean ▼ | CEM III/B ▼ | assign |
| Girder (3) | B400 ▼ | good ▼ | 35 mm ▼ | B1 mean ▼ | CEM III/B ▼ | assign |
| Abutment (1) | B225 ▼ | average ▼ | 25 mm ▼ | B5 mean ▼ | CEM I ▼ | assign |
| Abutment (2) | B225 ▼ | bad ▼ | 25 mm ▼ | B5 mean ▼ | CEM I ▼ | assign |
| Column (1) | B600 ▼ | bad ▼ | 45 mm ▼ | B7 max ▼ | CEM III/B ▼ | assign |
| Column (2) | B600 ▼ | good ▼ | 45 mm ▼ | B7 max ▼ | CEM III/B ▼ | assign |
| Edge beam - left (1) | B225 ▼ | bad ▼ | 25 mm ▼ | B1 min ▼ | CEM I ▼ | assign |
| Edge beam - left (2) | B225 ▼ | average ▼ | 25 mm ▼ | B1 min ▼ | CEM I ▼ | assign |
| Edge beam - left (3) | B225 ▼ | average ▼ | 25 mm ▼ | B1 min ▼ | CEM I ▼ | assign |
| Edge beam - right (1) | B225 ▼ | good ▼ | 25 mm ▼ | B1 min ▼ | CEM I ▼ | assign |
| Edge beam - right (2) | B225 ▼ | good ▼ | 25 mm ▼ | B1 min ▼ | CEM I ▼ | assign |
| Edge beam - right (3) | B225 ▼ | good ▼ | 25 mm ▼ | B1 min ▼ | CEM I ▼ | assign |

Fig. 4 Default mode of entering element specific material parameters in the LeCIE_tool



Abutment (A) ▼

*Originally selected concrete: B5 mean 25*

| Name | Description | Value | Unit | VP1 | VP2 | VP3 |
|---|---|---|---|---|---|---|
| $T_{ref}$ | Reference temperature | 80 | K | | | |
| $t_0$ | Reference point of time | 0.0767 | y | | | |
| $\Delta x$ | Depth of convection zone | 0 | mm | | | |
| $C_{S,\Delta x}$ | Chloride content at depth $\Delta x$ | 2.0 | wt.-%/c | | | |
| $T_{real}$ | Real temperature | 293 | K | | | |
| $C_0$ | Initial chloride content | 0.2 | wt.-%/c | | | |
| $x$ | Depth with a corresponding content of chloride ($x = c$) | 25 | mm | | | |
| $\alpha$ | Ageing exponent | 0.65 | - | | | |
| $b_e$ | Temperature coefficien | 4800 | K | | | |
| $D_{RCM,0}$ | Chloride migration coefficient (RCM test) | 15.6 | $m^2/s$ | | | |
| $C_{crit}$ | Critical chloride content | 0.6 | wt.-%/c | | | |

Enter change

Fig. 5 Detailed mode of entering values for each parameter of deterioration in the LeCIE_tool, case of chloride ingress in bridge abutment

a bridge in the LeCIE_tool can offer his judgment on the quality of execution, depth of concrete cover or environmental loading. By choosing the execution quality, the user adjusts the w/c ratio and thus indirectly the resistance properties of concrete. In the cases where substantial information on durability of particular bridge element exists, the users can use the detailed mode depicted in Fig. 5. The information to be used in detailed mode can be collected from different sources, such as design and execution documents, reports from previous inspections, repair reports, measurements, detailed testing, and so on.

In proceeding chapters, a more detailed explanation of material parameters for both modes will be given.

Table 2 Recommended environmental concrete classes according to ÖNORM B 4710-1 (2004) and thus covered exposure classes of EN 206-1 (2013)

| Designation | Covered exposure classes | w/c | Air content | Example |
|---|---|---|---|---|
| B1 | **XC3** | 0.60 | - | Water pressure < 10 m |
| **B2** | **XC3**/**XD2**/XF1/XA1L/SB | 0.55 | - | Swimming pools |
| **B3** | **XC3**/**XD2**/XF3/XA1L/SB | 0.55 | 2.5 – 5.0 | Horizontal hydraulic structures |
| B4 | XC4/XD2/XF1/XA1L/SB | 0.50 | - | Water pressure > 10 m |
| **B5** | **XC4**/**XD2**/XF2/XA1L/SB | 0.50 | 2.5 – 5.0 | Spray containing de-icing agents |
| B6 / $C_3A$ free | XC4/XD2/XF3/XA2L/ XA2T/SB | 0.45 | 2.5 – 5.0 | Sewage systems |
| **B7** | **XC4**/**XD3**/XF4/XA1L/SB | 0.45 | 4.0 – 8.0 | Direct de-icing agents |
| B8 | XC3/UB1 | 0.60 | - | Diaphragm walls (Slurry walls) |
| B9 | XC3/UB2 | 0.60 | - | Bored piles in dry conditions |
| B10 | XC3/XD2/XF1/XA1L/UB1 | 0.55 | - | Bored piles in water |
| B10 / $C_3A$ free | XC3/XD2/XF1/XA1L/XA1T/UB1/$C_3A$ free | 0.55 | - | Diaphragm wall: groundwater |
| B11 | XC3/XD2/XF1/XA1L/UB2 | 0.55 | - | Diaphragm wall: groundwater |
| B11 / $C_3A$ free | XC3/XD2/XF1/XA1L/XA1T/UB2/$C_3A$ free | 0.55 | - | Bored piles: groundwater |
| B12 | XC4/XD2/XF1/XA1L/UB1 | 0.50 | - | Bored piles: groundwater |
| B12 / $C_3A$ free | XC4/XD2/XF1/XA1L/XA1T/UB1/ $C_3A$ free | 0.50 | - | Diaphragm wall: water pressure > 10 m |
| HL-SW | XC4/XD3/XF3/XA3L/XA3T | 0.34 | - | Diaphragm wall: water pressure > 10 m |
| HL-B | XC4/XD3/XF4 | 0.34 | 4.0 – 8.0 | Diaphragm walls (Slurry walls) |

It is important to mention that in this moment, the LeCIE_tool can be applied only on deterministic bases. The implementation of the probabilistic approach, broadening the tool to other deterioration processes and implementation of other materials represent the next phases of development.

After assigning the deterioration parameters and performing the calculation of the deterioration path and remaining service life, the tool provides the optimization ofpossible maintenance and repair strategies with cost analysis based on the inputs from the ÖBB.

## 4. Exposure classes

The European Standard EN 206-1 (2013) divides the environmental conditions in exposure classes with descriptions and several informative examples. For the exposure class related to carbonation, designations from XC1 to XC4 are used, and for the chloride ingress the classes are divided in the XS class and the XD class. XS class represents corrosion induced by chlorides from seawater and XD presents corrosion induced by chlorides other than seawater. Since Austria is a landlocked country, only XD class is used, which can range from XD1 to XD3.

According to *fib* Bulletin 34 (2006), class XD1 is considered to be in the spray zone, while class XD3 is described as the splash zone. Contrary, EN 206-1 (2013) refers to XD1 class as airborne chloride exposure, and class XD3 as part of bridges exposed to spray containing chlorides. Moreover, it is by default assumed that all the elements at the distance larger than 1.5 m from the road have moderate humidity, which does not need to be the case.

Austrian standard ÖNORM B 4710-1 (2004): ‘Specification, production, use and verification of conformity’, which provides rules for the implementation of EN 206-1 (2013) for normal and heavy concrete, recommends a dozen of environmental concrete classes. The environmental concrete classes are chosen based on the environmental exposure, where maximum w/c, air content range and example of usage are recommended. For recommended concrete types, covered exposure classes from EN 206-1 (2013) are also assigned, as shown in the second column in Table 2.

Classes B1 and B4 are suitable for water impermeable components, while other environmental concrete classes from B2 to B7 are suitable for environmentally affected components. For the purposes of the tool, the B1 class is considered suitable for elements without chloride contamination, since it is the only environmental concrete class without XD exposure assigned. The environmental concrete classes from B8 onward are used for deep bridge foundations. In Table 2, concrete types relevant for this article are shaded in grey.

Representation of environmental concrete classes in Table 2 does not cover the exposure classes XC1, XC2 and XD1 from EN 206-1 (2013). Moreover, all the environmental concrete classes used, except B1 and B7, are considered to have the XD2 exposure class, which represents rarely dry concrete. Furthermore, some of the environmental concrete classes simultaneously represent both sheltered/dry elements for carbonation and wet/rarely dry conditions for chlorides. Since in literature, parameters of chloride and carbonation models are given dependent on the exposure classes, the stated incompatibilities create difficulties in assigning the model parameters to environmental concrete classes. Furthermore, while assigning the environmental concrete classes, one simultaneously assigns the maximum w/c ratio (thus, concrete quality) and the severity of exposure, which is more suitable for design than for determining service life of existing bridges. In other words, assigning the environmental concrete classes to existing bridges when performing the assessment will signify that a bridge in severe environmental exposure always has to have a good concrete quality, and vice versa. Examining the inspection records from existing bridges in ÖBB’s bridge stock, this presumption showed not always to be the case.

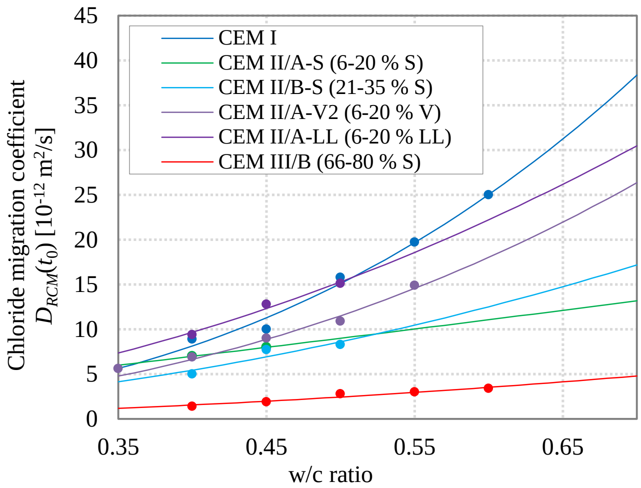



Fig. 6 Power law fitting of the chloride migration coefficient $D_{RCM,0}$ for different cement classes

## 5. Chloride ingress

A design approach for modelling chloride induced corrosion in uncracked concrete presented in this article is supported by the *fib* through the Model Code (2010) and Bulletins 34 (2006), 59 (2011) and 76 (2015). This approach is based on the limit state equation Eq. (1), where the critical chloride content is compared to the actual chloride content at the depth of reinforcing steel.

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

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

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

where:

$C_{crit}$ is the critical chloride content [wt.-%/c];
$C_0$ is the initial chloride content [wt.-%/c];
$C_{S,\Delta x}$ is the chloride content at depth $\Delta x$ [wt.-%/c];
$x$ is the depth with a corresponding content of chlorides ($c$ – depth of reinforcement) [mm];
$\Delta x$ is the depth of convection zone [mm];
$D_{app}(t)$ is the apparent chloride diffusion coefficient [m²/s];
$t$ is time [s].

For the quantification of the apparent chloride diffusion coefficient $D_{app}(t)$ two approaches can be found in literature. Herein, the approach based on the chloride migration coefficient is used.

Apparent chloride diffusion coefficient is

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

where:

$k_e$ is the environmental variable (considers temperature) [-];
$D_{RCM,0}$ is the chloride migration coefficient at the reference point of time (from RCM test) [m²/s];
$t_0$ is the reference point of time [y];
$\alpha$ is the ageing exponent (for approach with RCM test) [-].

Environmental variable is expressed as

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

where:

$b_e$ is the temperature coefficient (proportional to activation energy) [K];
$T_{ref}$ is the reference temperature [K];
$T_{real}$ is the temperature of the structural element or the ambient air [K].

In order to identify the values of delineated parameters of chloride ingress, an adjustment of properties of new modern to former concrete mixtures (classes) was required.

A detailed description of mapping the chloride transport characteristics and parameters used in the LeCIE_tool reaches beyond the scope of this article. Herein, only the most important parameters are shown and explained. For detailed insight in influence of the parameters to the outcome of the model and therefore to the durability performance of concrete structures, see Ferreira (2010).

### *5.1 Chloride migration coefficient – $D_{RCM,0}$*

Values of chloride migration coefficient $D_{RCM,0}$ for different cements used for the analysis in this article are based on *fib* Bulletin 76 (2015), and depicted in Fig. 6 with dots.

Although in different literature the chloride migration coefficient $D_{RCM,0}$ has been measured for a broader spectre of cement types (Gehlen 2000, Tang 1996, Tang *et al.* 2010), herein only the historically used cement types in Austria are presented (according to the Austrian standards ÖNORM B 3310 (1963, 1980, 1993)). In literature, the chloride migration coefficient $D_{RCM,0}$ has not been suggested for the entire w/c range (e.g., it was not suggested for w/c = 0.65, 0.70, etc.). Therefore, the power law fitting is performed in order to approximate the missing values, as shown with lines in Fig. 6. Although, such approximations can be found in similar contributions, e.g., in Vu and Stewart (2000), they should be omitted when real tested values of the chloride migration coefficient $D_{RCM,0}$ become available for the whole w/c range.

### *5.2 Concrete cover – c*

Concrete cover was tested on several bridges from Ö BB and the measurements were documented in test reports. Tests were conducted from 2008 to 2013, on bridges that were built between 1967 and 1984. These tests revealed high deviations of concrete cover among bridges, even those with similar age and structural systems. Due to existing variety among concrete covers, values of 25 mm, 35 mm and 45 mm are set to be chosen as default for the analysis in the tool.

### *5.3 Ageing exponent – α*

The ageing exponent α introduces the decrease of the chloride diffusion coefficient $D_{app}(t)$ in time (DARTS, 2004), and in a mathematical sense, it presents a slope in a double-logarithmic diagram. It is a parameter that is considered dependent on both material and environmental conditions. Classification of ageing exponents is also done on the bases of *fib* Bulletin 76 (2015) and historically used cement types in Austria. The values of the ageing exponent α implemented in the tool are as laid out in Table 3.

### *5.4 Chloride content – $C_{S,\Delta x}$*

Chloride content at the depth of the convection zone $\Delta x$ is a parameter that varies most when considering chloride transport parameters. The convection zone represents the depth of up to which the transport of chlorides differs from Fick's 2nd law of diffusion. The variations in $C_{S,\Delta x}$ are even more pronounced in the exposure classes related to de-icing salts. The cause of such variations lies in the different frequency of application of de-icing salts, concrete composition and humidity, chloride content in ambient solution, and so on (fib Bulletin 76, 2015). As for the concrete cover, the values of $C_{S,\Delta x}$ = 2, 3 and 4 [wt.-%/c] are offered as default, thus offering the engineer to choose a minimum, mean or maximum environmental loading.

### *5.5 Reference temperature – $T_{ref}$*

Chloride ingress is assumed to be a thermodynamic process (Yuan *et al*. 2008, Care 2008).

Table 3 The values of the ageing exponent α for the historically used cement types in Austria, extracted from *fib* Bulletin 76 (2015)

| CEMENT TYPE | Ageing exponent $\alpha$ [-] |
|---|---|
| CEM I | 0.30 |
| CEM II/A-S | 0.35 |
| CEM II/A-V | 0.60 |
| CEM II/A-LL | 0.30 |
| CEM III/B | 0.45 |

The influence of temperature in the model used in this article can be taken into account via the ambient temperature $T_{real}$. In order to determine the ambient temperature $T_{real}$, the average annual ambient temperature of the structure should be used. Such temperature can be obtained from nearest weather stations. For the needs of the LeCIE_tool, statistical calculations are performed based on measurements of temperature (ZAMG 2017). The mean values and standard deviations for every state in Austria are calculated. In this way, depending on the entered route and position of the bridge, the tool can automatically assign the corresponding mean temperature.

## 6. Carbonation

The carbonation formulation presented and utilized in this article is in the form given in *fib* Bulletins 34 (2006) and 59 (2011), and it was initially developed in the research projects DARTS (2004) and DuraCrete (1998). Later *et al.* (2016a) introduced the additional parameter – carbonation rate $k_{NAC}$, which replaced the inverse carbonation resistance $R_{NAC}^{-1}$. The limit state equation, where the carbonation depth is compared with concrete cover is given by Eq. (5).

$$g(t) = c - x_c(t) \tag{5}$$

where:
$x_c(t)$ is the carbonation depth at time $t$ [mm].

Further, carbonation depth can be expressed as

$$x_c(t) = k_{NAC} \cdot \sqrt{k_e \cdot k_c \cdot k_a} \cdot \sqrt{t} \cdot W(t) \tag{6}$$

where:
$k_{NAC}$ is the carbonation rate for standard test conditions [mm/years$^{0.5}$];
$k_e$ is the function describing the environmental effect of relative humidity $RH_a$;
$k_c$ is the function describing the effect of curing/execution;
$k_a$ is the function describing the effect of $CO_2$ concentration in the ambient air;
$W(t)$ is the function describing the effect of wetting events.

Environmental coefficient $k_e$ is described as

$$k_e = \left( \frac{1 - \left(\frac{RH_a}{100}\right)^{f_e}}{1 - \left(\frac{RH_l}{100}\right)^{f_e}} \right)^{g_e} \tag{7}$$

where:
$RH_a$ is the relative humidity of ambient air [%];
$RH_l$ is the reference humidity [%];
$f_e$ is the exponent [-];
$g_e$ is the exponent [-].

Curing/execution coefficient $k_c$ is

$$k_c = \left(\frac{t_c}{7}\right)^{b_c} \tag{8}$$

where:
$t_c$ is the curing time [days];
$b_c$ is the exponent [-].

Table 4 Overall parameters of chloride ingress implemented in the default mode of the LeCIE_tool

| | | | B1 | B2 | B3 | B5 | B7 |
|---|---|---|---|---|---|---|---|
| Parameter | | Unit | | | Value | | |
| Water-cement ratio | w/c | - | 0.57 | 0.52 | 0.50 | 0.45 | 0.38 |
| Reference temperature | $T_{ref}$ | K | | | 293 | | |
| Temperature of the structural element | $T_{real}$ | K | | | 281 | | |
| Temperature coefficient | $b_e$ | K | | | 4800 | | |
| Initial chloride content | $C_0$ | wt.-%/c | | | 0.10 | | |
| Critical chloride content | $C_{crit}$ | wt.-%/c | | | 0.60 | | |
| Ageing exponent | α | - | | | Table 3 | | |
| Chloride content at the depth $\Delta x$ | $C_{S,\Delta x}$ | wt.-%/c | 0 | | 2 / 3 / 4 | | |
| Concrete cover | $c$ | mm | | | 25 / 35 / 45 | | |
| Depth of the convection zone | $\Delta x$ | mm | 0 | 0 | 0 | 0 | 9 |
| Chloride migration coefficient | $D_{RCM,0}$ | m²/s | | | Fig. 6 | | |

$CO_2$ concentration coefficient $k_c$

$$k_a = \frac{C_a}{C_l} \tag{9}$$

where:

$C_a$ is the $CO_2$ concentration of ambient air [kg/m$^3$] or [vol. %];

$C_l$ is the $CO_2$ concentration during concrete testing [kg/m$^3$] or [vol. %].

The environmental/wetting function $W(t)$

$$W(t) = \left(\frac{t_0}{t}\right)^{\frac{(p_{dr} \cdot ToW)^{bw}}{2}} \tag{10}$$

where:

$p_{dr}$ is the probability of driving rain [-];

$ToW$ is the time of wetness [-], i.e. days with daily rainfall ≥ 2.5 mm;

$b_w$ is the regression exponent [-].

### 6.1 Carbonation rate – $k_{NAC}$

Von Greve-Dierfeld and Gehlen tested the carbonation rates of different concrete mixtures under standard test conditions (von Greve-Dierfeld and Gehlen 2016a, b, c). Such obtained carbonation rates are transformed in order to relate the carbonation rate $k_{NAC}$ to a specific combination of cement class and w/c ratio. As in chlorides, here also, values of carbonation rate $k_{NAC}$ for some w/c ratios were not available in literature, and therefore they were linearly extrapolated, as depicted in Fig. 7. In Fig. 7, the dots represent measured values while the lines represent linear extrapolation.

### 6.2 Curing time – $t_c$

The effect of curing is one of prevailing effects in the analysis of carbonation, and it is described by the curing/execution coefficient $k_c$ shown in Eq. (8), in which the curing time $t_c$ is taken into account. For the experimental results of $k_c$, see Sisomphon and Franke (2007) and Hunkeler (2012). Generally, in the past, curing time was shorter than the one prescribed today. On the bases of VÖZ - Association of Austrian Cement Industry (1985), where curing times in the past are suggested, a 3-day curing time is chosen for the default mode.

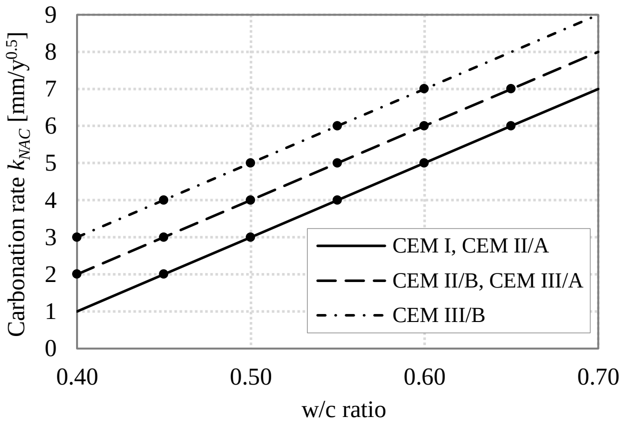



Fig. 7 Carbonation rate for standard test conditions approximated with linear function extracted from von Greve-Dierfeld and Gehlen (2016c)

### 6.3 $CO_2$ concentration

The $CO_2$ concentration is taken into account through $k_a$ – a function describing the effect of $CO_2$ concentration in ambient air. Coefficient $k_a$ shows the relation of ambient air $CO_2$ content $C_a$ and the laboratory $C_l$ content, used for the standard test purposes, and as such is dimensionless. Although von Greve-Dierfeld and Gehlen (2016a) state that the $CO_2$ exposure is essentially independent of location, in the LeCIE_tool it was varied between values of 0.036, 0.040 and 0.044, thus, offering the possibility of differentiation regarding bridges in rural and urban areas.

## 7. Preliminary results of the LeCIE_tool – default mode

In order to perform a verification of the analysis tool through a prediction of the remaining service life, a database of 17 ÖBB bridges is used. The database mostly comprises single span concrete bridges with length of up to 37 m. The bridge elements are between 2 and 82 years old, with mean value of 44.2 years and standard deviation of 18.2 years. To capture similar environmental exposure, most of the bridges selected are located near Vienna, in the state of Lower Austria.

Table 5 Overall parameters of carbonation implemented in the default mode of the LeCIE_tool

| | | | B1 | B2 | B3 | B5 | B7 |
|---|---|---|---|---|---|---|---|
| Parameter | | Unit | Value | | | | |
| Water-cement ratio | w/c | - | 0.57 | 0.52 | 0.5 | 0.45 | 0.38 |
| Relative humidity - laboratory | $RH_l$ | % | | | 65 | | |
| Exponent | $f_e$ | - | | | 5 | | |
| Exponent | $g_e$ | - | | | 2.5 | | |
| Curing time | $t_c$ | days | | | 3 | | |
| Regression exponent | $b_w$ | - | | | 0.446 | | |
| Exponent | $b_c$ | - | | | -0.567 | | |
| $CO_2$ concentration of ambient air | $C_a$ | vol. % | | | 0.036/ 0.040/ 0.044 | | |
| $CO_2$ concentration - laboratory | $C_l$ | vol. % | | | 0.04 | | |
| Concrete cover | $c$ | mm | | | 25/ 35/ 45 | | |
| Carbonation rate | $k_{NAC}$ | mm/years$^{0.5}$ | | | Fig. 7 | | |
| Relative humidity of ambient air | $RHa$ | % | | | 80 | | |
| Time of wetness | $ToW$ | % | 0.0 | 0.0 | 0.0 | 0.20 | 0.20 |
| Probability of driving rain | $p_{dr}$ | % | 0.0 | 0.0 | 0.0 | 0.1 | 0.1 |

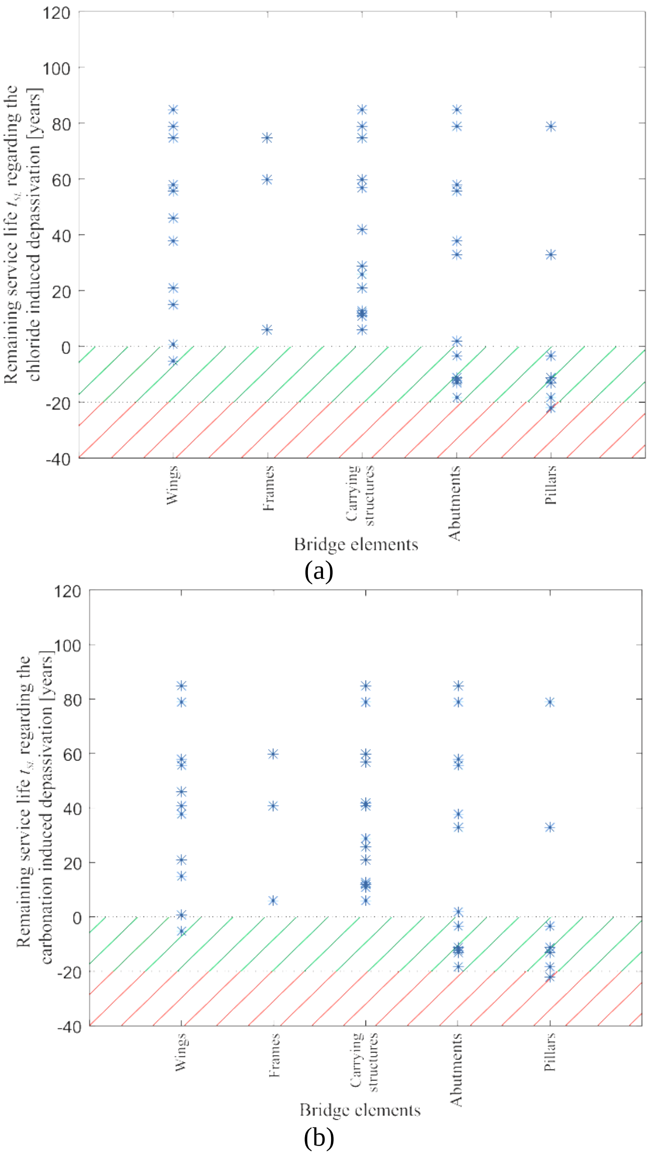



Fig. 8 LeCIE_tool database results for the prediction of remaining service life for a dozen of test bridges from Austrian Federal Railways, regarding depassivation caused by (a) chloride ingress and (b) carbonation

Herein, for the prediction of the remaining service life, only the values implemented in the default mode of the LeCIE_tool are used. Varying default parameters from Tables 4 and 5 are taken as $c$ = 35 mm, $C_a$ = 0.04 vol. % and $C_{S,\Delta x}$ = 2.0. Moreover, preliminary results are given taking into account the deterioration values on the example of cement type CEM II/A-S, as one of several cement types used in Austria since 1963 (Ö NORM B 3310, 1963).

Chosen parameters are implemented in the tool and the remaining service life regarding the depassivation caused by carbonation and chloride ingress is calculated. For the elements with B1 environmental concrete class, meaning they are not exposed to chlorides, a maximum service life of 120 years is envisaged. Therefore, for bridge elements with the B1 class, remaining service life was based on the revocation of the current age of an element from the maximum service life. The results of calculation of the remaining service life are shown in Fig. 8, for each element separately. The values that suggest that certain elements are already depassivated, meaning the remaining service life is negative, are indicated by lines.

## 8. Case study – Weickendorf railway bridge

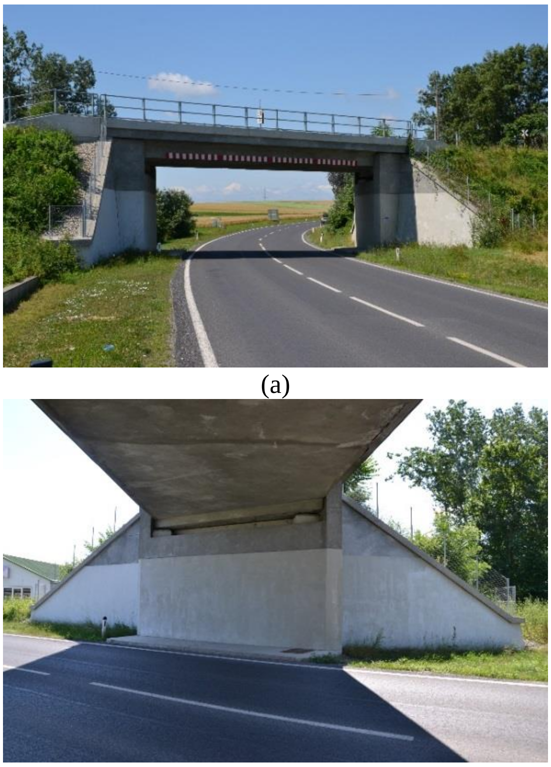
A case study of the Weickendorf railway bridge is used for a comprehensive description and elaboration of the detailed mode of the LeCIE_tool. The Weickendorf railway bride (database bridge number: 4005026) is an Austrian Federal Railway single span bridge, which was built in 1967 on the route 1151-33003 Gänsendorf – Marchegg in the district of Gänsendorf, state of Lower Austria. The bridge girder is a reinforced concrete plate, approximately 19 m long and 7 m wide. The bridge consists of standard bridge elements such as abutments, wings, plate girder, side beams, bearings etc., whose design and dimensions can be found in Grossberger (2014).

During the standard periodical visual inspection of the bridge in 2013, at the age of 46 years, several damage zones were detected with spalled parts of concrete, which triggered additional testing of chloride content and carbonation depth. Tests revealed high content of chlorides and significant carbonation in several bridge elements. Therefore, it was decided that certain parts of the bridge were to be repaired. The condition of the bridge after the repair can be seen in Fig. 9.

### 8.1 Testing campaign

The testing campaign consisted of determining the concrete compressive strength, carbonation depth, pH-value, chloride concentration and reinforcement depth. Cores for determining the compressive strength were drilled on abutments; chloride concentration was measured on abutments and wings; concrete cover was measured only on abutments; carbonation was determined on abutments, wings, girder and edge beams.

According to the design documents, the abutments were designed as the B160 concrete strength class (ÖNORM B 4710 (2004)), which would correspond to modern day concrete C12/16. Contrary to that, tests have shown significantly higher values of compressive strength compared to design, with an average compressive strength of 39.4 MPa. When measured strengths are associated with VÖZ (1985) (where recommendations for w/c ratio in former strength classes are given), w/c ratio of approximately 0.45-0.55 can be presumed to have been used in the abutments. In cases when sufficient and reliable measured data on concrete strength exists, a recalculation of strength needs to be performed, since the properties of deterioration models applied in this article are based on concrete age of 28 days.



(a) (b)

Fig. 9 Case study - bridge 4005026 after repair 2013; (a) side view and (b) bottom view

Table 6 Results of testing carbonation depth $x_c(46)$ on bridge 4005026 for abutments (A), wings (W), edge beam (EB) and deck (D)

| Point | Height (m) | Average value (mm) | Extreme value (mm) |
|---|---|---|---|
| A 1 | 0.20 | 38 | 41 |
| A 2 | 1.30 | 49 | 51 |
| A 3 | 2.35 | 52 | 53 |
| A 4 | 3.40 | 20 | 23 |
| A 5 | 3.40 | 18 | 21 |
| A 6 | 2.35 | 25 | 28 |
| A 7 | 1.30 | 46 | 51 |
| A 8 | 0.20 | 37 | 39 |
| W 1 | 1.50 | 15 | 22 |
| W 2 | 1.50 | 23 | 28 |
| W 3 | 1.50 | 16 | 21 |
| W 4 | 1.50 | 46 | 47 |
| EB 1 | > 4.00 | 1 | 2 |
| D 1 | > 4.00 | 1 | 2 |

Table 7 Results of chloride content $C(c,46)$ [wt.-%/c] testing on bridge abutments (A) and wings (W), at depth of 15, 30 and 45 mm

| Point | Height (m) | Chloride concentration $C(c,46)$ [wt.-%/c] | | |
|---|---|---|---|---|
| | | 0-15 (mm) | 15-30 (mm) | 30-45 (mm) |
| A 1 | 0.20 | 2.20 | 2.20 | 1.20 |
| A 2 | 1.30 | 0.87 | 1.10 | 0.24 |
| A 3 | 2.35 | 0.22 | 0.28 | 0.20 |
| A 4 | 3.40 | 0.21 | 0.00 | 0.00 |
| A 8 | 0.20 | 3.90 | 2.30 | 2.50 |
| A 7 | 1.30 | 1.10 | 1.20 | 0.87 |
| A 6 | 2.35 | 0.37 | 0.41 | 0.18 |
| A 5 | 3.40 | 0.24 | 0.00 | 0.00 |
| W 1 | 1.50 | 0.21 | 0.19 | 0.15 |
| W 2 | 1.50 | 0.27 | 0.26 | 0.31 |
| W 3 | 1.50 | 0.48 | 1.10 | 0.59 |
| W 4 | 1.50 | 0.00 | 0.11 | 0.48 |

Concrete cover was measured only on abutments, where horizontal reinforcement had the cover of circa 30 mm and vertical of circa 40 mm. Horizontal reinforcement has a diameter ϕ14 on a raster of 15 cm, while vertical reinforcement has a diameter ϕ12 on a raster of 20 cm.

Carbonation was measured on the same vertical heights as chloride, as presented in Table 6 and Fig. 7. The results of carbonation depth $x_c(46)$ for abutments (A), wings (W), edge beam (EB) and deck (D) are given as average and extreme values.

### 8.2 Results of the detailed mode

The values obtained in the testing campaign at the age of 46 years given in the previous chapter are used to adjust the values in the LeCIE_tool default mode, through the input interface presented in Fig. 5. The values that are adjusted are stated in Figs. 10 and11. Other input parameters used for the prediction are taken as in default mode and are based on the values shown in Tables 4 and 5.

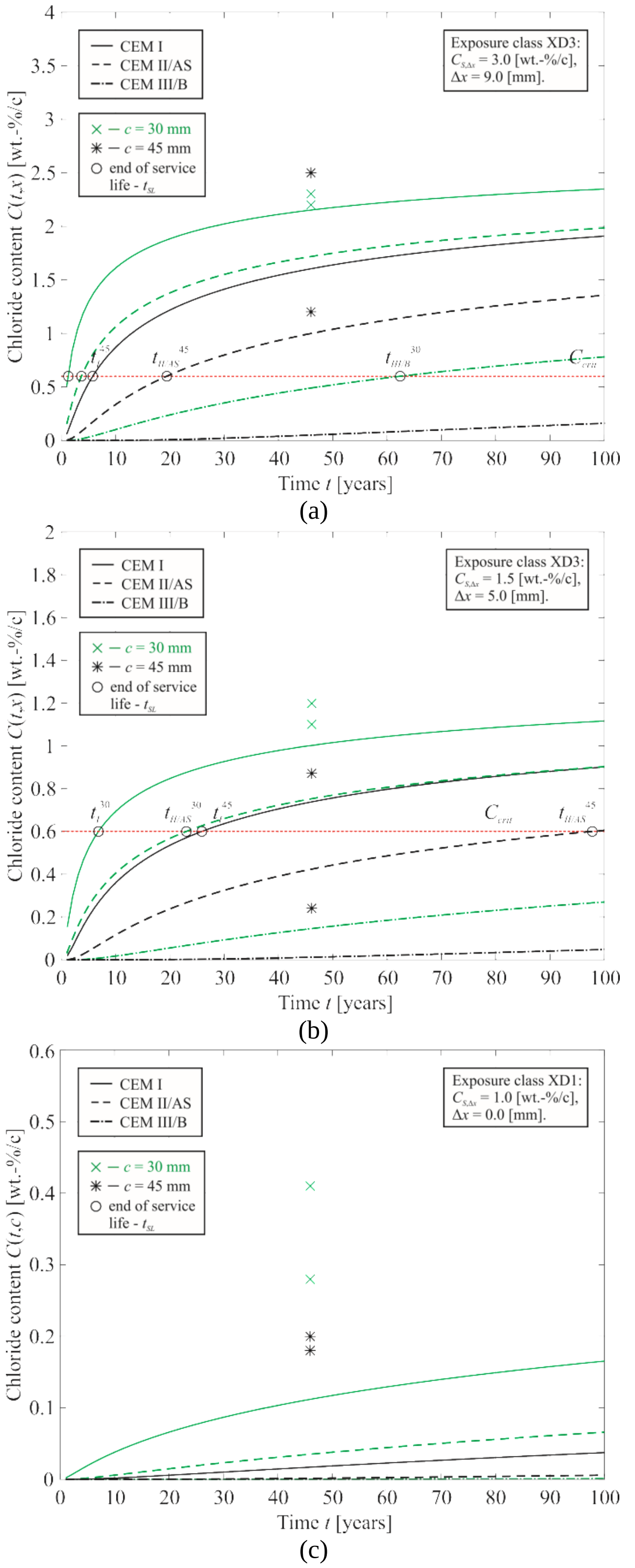



Fig. 10 Calculation of chloride content $C(c,t)$ and service life duration $t_{SL}$ based on cement types CEM I, CEM II/A-S, and CEM III/B, for bridge abutments at heights of (a) 0.20 m, (b) 1.30 m, and (c) 2.35 m

Table 8 Resistivity parameters of chloride ingress and carbonation for different cement types, based on the presumed w/c ratio 0.5

| Cement type | $D_{RCM,0}$ [$m^2$/s] | α [-] (XD3/XD1) | $k_{NAC}$ [mm/$y^{0.5}$] |
|---|---|---|---|
| CEM I | 15.8 | 0.30 / 0.65 | 3.0 |
| CEM II/A-S | 9.0 | 0.35 / 0.65 | 4.0 |
| CEM III/B | 2.8 | 0.45 / 0.65 | 5.0 |

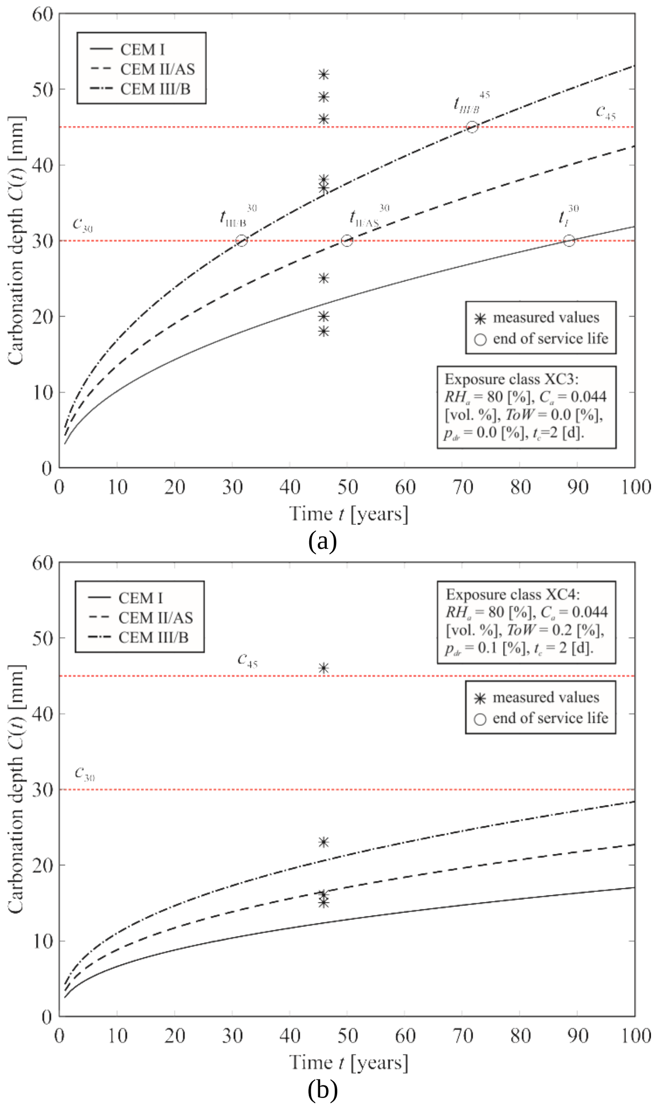



Fig. 11 Calculation of carbonation depth $x_c(t)$ and service life duration $t_{SL}$ based on cement types CEM I, CEM II/A-S, and CEM III/B for (a) abutments and (b) wings

Since the test report has not provided exact values of measured concrete cover, just approximate observations, the analysis is performed with covers of 30 and 45 mm. In this way, it is possible to compare the theoretical results derived from calculation with measured values of chloride concentration. Furthermore, since the exact cement type used in the bridge is unknown, the analysis of chloride content and carbonation depth is performed on three different cement types, all used in Austria at the time of construction. Namely, the used cement types are CEM I, CEM II/A-S and CEM III/B. Resistivity parameters for chosen cement types are shown in Table 8, and are based on the presumed w/c ratio 0.5.

In Fig. 10, a point in time $t_{SL}$, in which the chloride concentration becomes larger than the critical chloride concentration $C_{crit}$, is depicted, thus presenting the time in which an element ends its service life regarding the limit state of chloride-induced depassivation. For carbonation, depicted in Fig. 11, $t_{SL}$ represents a point in time in which the carbonation depth $x_c(t)$ becomes larger than the concrete cover $c$. In Figs. 10 and 11 the duration of service life $t_{SL}$ is depicted for different cement types and concrete covers as $t_{cem}^{c}$, where $c$ represents the concrete cover and $cem$ is cement type.

Specific input parameters used for calculation are stated in Figs. 10 and 11. Other input parameters for chloride ingress can be found in Table 4, whereas Table 5 presents carbonation parameters. Along with calculated (theoretical) values, the measured values are also shown for both the carbonation and chloride ingress, as given in Tables 6 and 7, respectively.

In Fig. 10, green lines depict calculated values for concrete depth of 30 mm, and black lines represent concrete cover of 45 mm. Moreover, chloride levels measured at 30 mm depth are depicted with the green X mark and the ones measured at 45 mm with the black asterisk symbol.

The calculated carbonation depth $x_c(46)$ and subsequent service life duration $t_{SL}$ are presented for abutments and wings separately, due to a different exposure situation, as depicted in Fig. 11. Measured values of carbonation in Fig. 11 are marked with the black asterisk symbol.

## 9. Discussion on derived results

In Fig. 10, a great scatter among measured values can be seen for abutments when same heights and depths are considered. Moreover, as it can also be seen in Table 7, the change of measured chloride content in depth is not homogeneous as it could have been expected. When comparing measured and calculated values of chloride content, one can easily notice a deviation for height of 2.35 m. In Fig. 11, a wider scatter of calculated results among different cement types is more evident in exposure class XD3 than in XD4. In addition, a generally fair matching of calculated and measured results of carbonation depths can be seen, except for a single measured value on wings that stands out from the average.

In general terms, the majority of calculated results show lower values than measured. Since the formulation of both chloride ingress and carbonation is formed on the basis of numerous parameters, these deviations could be a product of underestimation of certain crucial parameters, such as $C_{S,\Delta x}$ and $C_a$.

Considering the service life duration $t_{SL}$ of chlorides, abrupt ingress in the early stage causes the element to reach its service life early on (during the first 10 years) or, scarcely, at a later stage. As for the carbonation in exposure class XC4 in Fig. 11(b), the service life of wings does not reach its end, possibly due to the beneficiary effect of the wetting periods.

Two measurements exist per location when considering chloride content (measured at same height and depth), thus permitting the mean value μ to be determined. Contrary to chloride content, more than two measurements per location exist for carbonation, making it possible to determine the mean value μ and standard deviation σ. Hence, mean values and standard deviations are calculated for abutments and wings, and presented in Table 10. Furthermore, Tables 9 and 10 show the service life duration $t_{SL}$, where the service life duration of carbonation is divided in $t_{SL}^{35}$ and $t_{SL}^{45}$, creating a distinction between concrete cover of 35 and the one of 45 mm. With the aim of additionally clarifying the differences of calculated and measured values, Tables 9 and 10 present absolute deviations marked as $D_C(c,46)$ for chloride content and $D_{x,c}(46)$ for carbonation. These deviations are obtained as shown in Eq. (11)

$$D = |x_{cal} - \mu_{mea}| \tag{11}$$

where:

$D$ is the absolute deviation;
$x_{cal}$ is the calculated value;
$\mu_{mea}$ is the mean value of measured results.

Based on statistical moments of measured carbonation depth in Table 10, it is possible to construct a probability distribution. Herein, both the normal and lognormal distribution is assigned to measured values, as presented in Figs. 12 and 13.

Table 9 Comparison of calculated values $C(c,46)_{cal}$ and measured values $C(c,46)_{mea}$ of chloride content, for bridge abutments for different concrete covers and of diverse cement types, at heights (a) 0.20 m, (b) 1.30 m, and (c) 2.35 m

| $c$ [mm] | Cement type | $t_{SL}$ [y] | $C(c,46)_{cal}$ [wt.-%/c] | $C(c,46)_{mea}$ [wt.-%/c] | $D_C(c,46)$ [wt.-%/c] |
|---|---|---|---|---|---|
| (a) | | | | | |
| 30 | CEM I | 1 | 2.15 | μ = 2.25 | 0.10 |
| | CEM II/A-S | 4 | 1.72 | | 0.53 |
| | CEM III/B | 62 | 0.49 | | 1.76 |
| 45 | CEM I | 6 | 1.60 | μ = 1.85 | 0.25 |
| | CEM II/A-S | 19 | 1.00 | | 0.85 |
| | CEM III/B | > 100 | 0.05 | | 1.80 |
| (b) | | | | | |
| 30 | CEM I | 7 | 1.00 | μ = 1.15 | 0.15 |
| | CEM II/A-S | 23 | 0.75 | | 0.40 |
| | CEM III/B | < 100 | 0.15 | | 1.00 |
| 45 | CEM I | 26 | 0.73 | μ = 0.56 | 0.17 |
| | CEM II/A-S | 98 | 0.42 | | 0.14 |
| | CEM III/B | < 100 | 0.01 | | 0.55 |
| (c) | | | | | |
| 30 | CEM I | < 100 | 0.19 | μ = 0.35 | 0.16 |
| | CEM II/A-S | < 100 | 0.03 | | 0.32 |
| | CEM III/B | < 100 | < 0.01 | | > 0.34 |
| 45 | CEM I | < 100 | 0.02 | μ = 0.19 | 0.17 |
| | CEM II/A-S | < 100 | < 0.01 | | > 0.19 |
| | CEM III/B | < 100 | < 0.01 | | > 0.19 |

Table 10 Comparison of calculated values $x_c(46)_{cal}$ and measured values $x_c(46)_{mea}$ of carbonation depth for different concrete covers and cement types, for (a) abutments and (b) wings

| Cement type | $t_{SL}^{35}$ [y] | $t_{SL}^{45}$ [y] | $x_c(46)_{cal}$ [mm] | $x_c(46)_{mea}$ [mm] | $D_{x,c}(46)$ [mm] |
|---|---|---|---|---|---|
| (a) | | | | | |
| CEM I | > 100 | > 100 | 21.6 | | 14.00 |
| CEM IIAS | 68 | > 100 | 28.8 | μ = 35.6<br>σ = 13.3 | 6.80 |
| CEM IIIB | 43 | 72 | 36.0 | | 0.40 |
| (b) | | | | | |
| CEM I | > 100 | > 100 | 12.4 | | 12.6 |
| CEM IIAS | > 100 | > 100 | 16.4 | μ = 25.0<br>σ = 14.4 | 8.60 |
| CEM IIIB | > 100 | > 100 | 20.6 | | 4.40 |

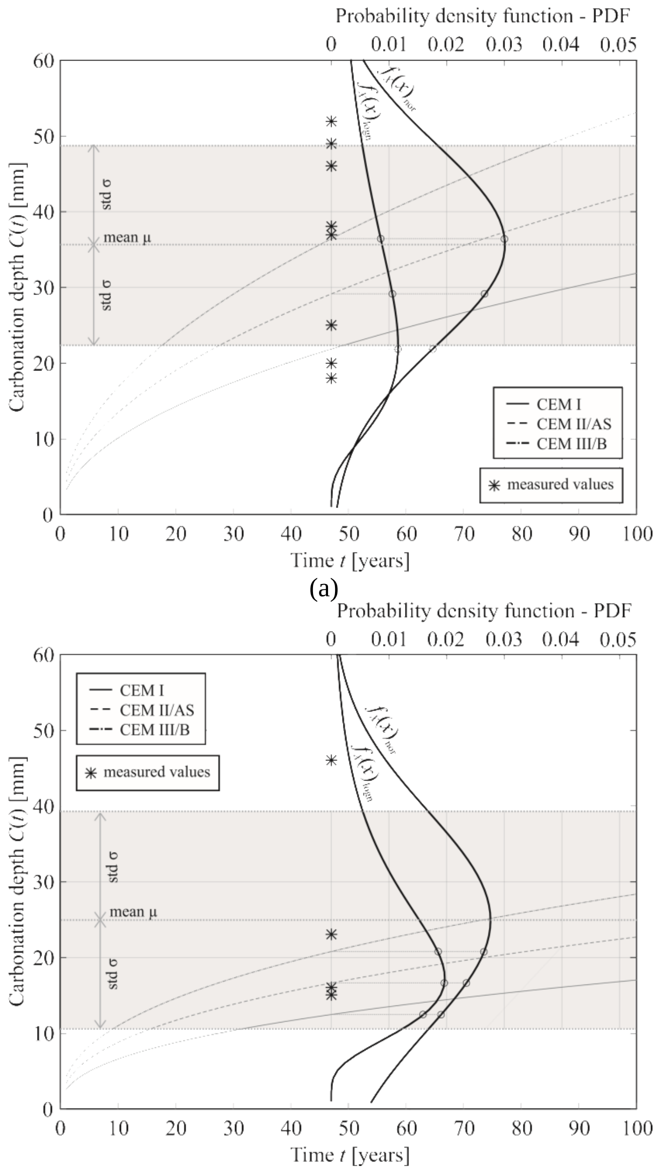



Fig. 12 Probability density functions (PDFs), based on measured depth of carbonation for (a) abutments and (b) wings

The probability density function (PDF) and cumulative distribution function (CDF) of normal distribution are given in Eqs. (12) and (13), respectively.

$$f_X(x) = \frac{1}{\sqrt{2\pi\sigma^2}} e^{-\frac{(x-\mu)^2}{2\sigma^2}} \tag{12}$$

$$F_X(x) = \frac{1}{2}\left[1 + \mathrm{erf}\left(\frac{x-\mu}{\sigma\sqrt{2}}\right)\right] \tag{13}$$

where:

$f_X(x)$ is the probability density function;
$F_X(x)$ is the cumulative distribution function;
μ is the mean or expectation of distribution;
σ is the standard deviation.

The PDF and CDF of lognormal distribution are given in Eqs. (14) and (15), respectively.

$$f_X(x) = \frac{1}{x\sigma\sqrt{2\pi}} e^{-\frac{(\ln x-\mu)^2}{2\sigma^2}} \tag{14}$$

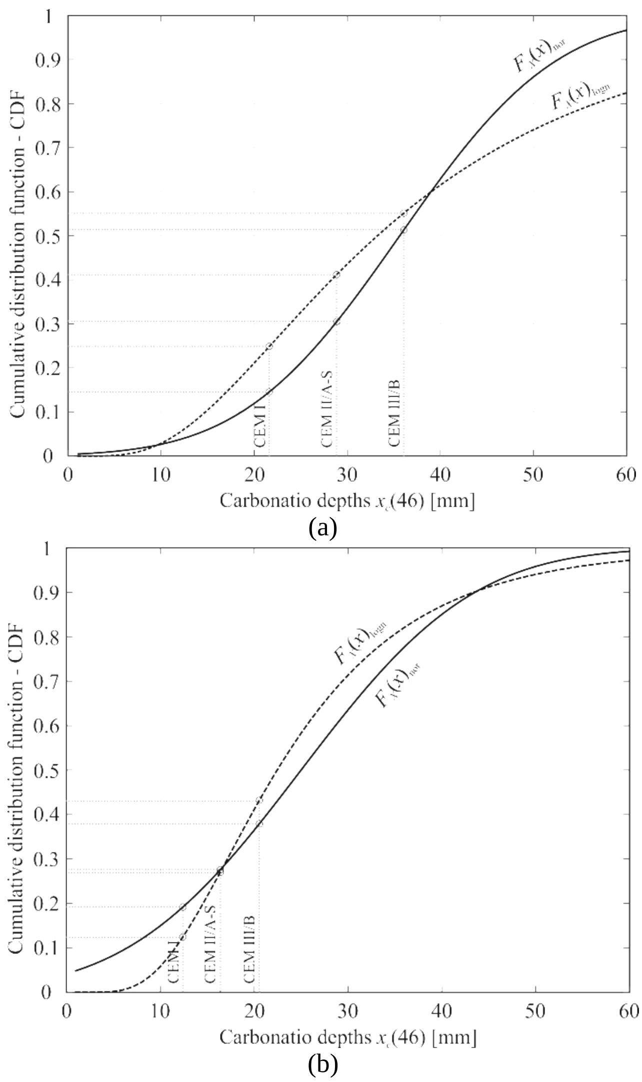



Fig. 13 Cumulative distribution functions (CDFs), based on measured depth of carbonation for (a) abutments and (b) wings

$$F_X(x) = \frac{1}{2} + \frac{1}{2}\operatorname{erf}(\frac{lnx - \mu}{\sigma\sqrt{2}}) \quad (15)$$

In Fig. 12, the PDF is shown for both distributions along with measured values and result of calculation. When referring to the CDF of measured values, the comparison of calculated and measured values is presented in Fig. 13, where calculated results are sorted by cement type used, i.e., CEM I, CEM II/A-S, and CEM III/B.

With the exception of one value, all other calculated results for carbonation $C(c,46)_{cal}$ fall within the standard deviation range of measured values, as shown in Fig. 12. Moreover, Figs. 12 and 13 depict points in which calculated values intersect with the PDFs and CDFs of measured values.

## 10. Conclusions

This article presents a prediction procedure of the remaining service life of existing concrete bridges in the network, based on the chloride- and carbonation-induced depassivation. The procedure is implemented in the BMS called the LeCIE_tool, currently being developed. The verification of the generated BMS is performed by predicting the remaining service life of elements of 17 bridges along with a comprehensive case study.

Main obstacle encountered while developing the LeCIE_tool was the identification of model parameters suggested in the international literature and their transformation to specific practice and understanding of environmental classes in Austria. When considering the crucial parameters of models, they can be available to engineers in different quality. For this reason, their implementation in the LeCIE_tool was divided into two different modes of input. These modes are the default mode and the detailed mode, and they distinguish bridges where no or few information is known and bridges where additional information is known based on measurements or tests.

Generally, the crucial parameters of the models should be studied in more detail, so potential errors, for instance the ones emerging from approximations performed in Figs. 6 and 7, could be avoided. Moreover, properties of concrete such as the exact type of cement and w/c ratio are hard to determine for existing concrete structures, especially in cases where they are not documented in design documents. Thus, in order to enable the use of the chloride ingress and carbonation models in the management of existing structures, documents such as the birth certificate, where durability related properties of newly built bridges are documented, should be strongly supported.

It should be emphasized that uncertainties play a significant role in determining service life duration, therefore prioritizing probabilistic and semi-probabilistic approaches over deterministic ones. However, the deterministic approach is deemed more simplistic and closer to engineers and technicians performing evaluation and management of existing bridges. Consequently, the BMS system, with its background described in this article, was initially commissioned as being based on the deterministic approach. The implementation of the probabilistic approach, broadening the tool to other deterioration processes and implementation of other materials represent the next phases of development.

According to calculation presented in this article, some of the bridge elements are already depassivated or should soon become depassivated. It can be said that these results are in accordance with the real situation, since several of the 17 bridges used in the calculation have already been repaired. The same conclusion can be extended to a case study example.

When compared to the majority of bridge management systems developed worldwide, the specificity of the LeCIE_tool is the way of predicting the future condition and remaining service life. The prediction of condition in the LeCIE_tool is modelled through physical models, contrary to the commonly used Markovian based models that are statistical in nature. The disadvantage of statistical models is their dependency on the quality of visual inspection input data. On the other hand, basic prediction with physical models does not depend on condition ratings from visual inspections and it always considers the particular deterioration phenomenon. The negative sides are the issues in determining exact parameters of each bridge and a high sensitivity of results to input parameters, which causes variations in the determined service life with a minor change of crucial parameters.

Regarding the predicted level of damage, these two methods of condition prediction significantly differ. Modelling with statistical models takes into account the damage described with condition ratings, which usually corresponds to the urgency of repair and stretches until the critical damage that can be considered as structural failure. Contrary to that, physical models of chloride ingress and carbonation given in *fib* Bulletins 34 (2006), 59 (2011) and 76 (2015) are focused on the limit state of depassivation, which is the serviceability limit state in which damage did not still occur. Thus, since these models are not connected with the formulation for corrosion propagation, they do not predict the occurrence of real damage and as such are more adequate for preventive than reactive maintenance.

Some of the case study calculated results show significant deviation compared to measured values. Such deviations can occur because of numerous parameters of chloride and carbonation formulations, which are difficult to determine with sufficient accuracy. Moreover, service life duration is difficult to predict accurately, since ambient conditions of a particular element are rarely stable. Hence, the significance of physical deterioration models within the BMS scope is not in their highly accurate predictability, but in creating a broader overview of the bridge stock condition with regard to a particular deterioration phenomenon. Once implemented, the approach should raise general awareness among infrastructure operators regarding topics such as crucial parameters of deterioration and importance of documenting the same. Furthermore, contrary to current bridge management practice, environmental loads and material parameters are taken into account, thus accentuating elements and structures which require more consideration. Therefore, it can be concluded that physical models such as chloride ingress and carbonation, in combination with findings of visual inspections and simple tests, present suitable means for the service life prediction in preliminary bridge assessment.

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