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ORIGINAL ARTICLE

Evaluation of service life design models on concrete structures exposed to marine environment

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Abstract Durability of reinforced concrete is primarily influenced by the penetration of aggressive substances into concrete, which are degrading concrete and reinforcement. For structures in marine environment chlorides are the most critical environmental load, which are causing serious corrosion damages. Data collected during the survey of the Krk Bridge, a large reinforced concrete arch bridge structure located on the Adriatic coast, is used as documented reference in this research. The structure has been exposed to the marine environment for over 25 years. Based on collected materials data and the exposure conditions, the service life of this structure is estimated using three currently available prediction models, two deterministic models, the North American *Life-365* model and the Croatian *CHLODIF* model, and the *DuraCrete* probabilistic method. All these models are based on the chloride diffusion process, but with different detailing of the model parameters. The conclusion is an evaluation of the service life predictive ability of each of these three service life models.

Keywords Chlorides · Reinforced concrete · Marine environment · Deterministic modelling · Probabilistic modelling

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1 Introduction

Durability of reinforced and prestressed concrete structures depend firstly on concrete penetrability, i.e. resistance to penetration of aggressive substances from the environment. In the last decades it became obvious that the corrosion of reinforcement is the most harmful damage which occurs either due to chloride attack or due to carbonation. The case study described in this paper is an example of a reinforced concrete structure exposed to a very aggressive marine environment for more than 25 years.

For structures in marine environment chlorides are the most critical environmental load, which are causing serious corrosion damages. The process of chloride penetration into the concrete is, therefore, one of the most important parameters for determining the service life of the structure; which can be theoretically determined by mathematical modelling of a chloride transport mechanism.

2 Scope of the research work

For the purposes of this research work, deterministic and probabilistic approaches to the calculation of the service life of structures were used [1]. The deterministic approach included the application of the existing numerical models: the North American *Life-365* and the Croatian *CHLODIF*, while the probabilistic approach was based on the *DuraCrete* model. The latter approach was developed within the scope the European project entitled "Probabilistic Performance Based Durability Design of Concrete Structures", or *DuraCrete* for short, completed in 1999, in which a full probabilistic method was developed for the calculation of structural service life following the same basic methodology as used for calculating the bearing capacity of structures [2]. All three models are based on the physical law of chloride diffusion.

The experimental part of this research work presents chloride analysis performed on the Krk Bridge, a spectacular reinforced concrete arch structure located on the Adriatic coast, that has been exposed to chloride action for more than 25 years. On the basis of the collected data, a statistical analysis was made for the purpose of determining the dependence of structural serviceability on the exposure zone and material's parameters.

The chloride profiles calculated following *Life-365* and *CHLODIF* were compared with the real in-field profiles of chlorides measured on the structure.

With the *DuraCrete* model the probability of corrosion was calculated as a function of time. The acceptable failure probability depends on the severity of the damage [3, 4].

The input parameters used for the calculations were the test data and the theoretically assumed values adopted from the manuals of the three models, such as diffusion coefficient and aging factor. In this paper the existing models for chloride ingress and their ability to predict the service life based on the deterministic and probabilistic approaches have been evaluated. Finally, steps are suggested for further development of the models with the aim to standardize the process of calculating the service life of structures exposed to the actions of chloride induced reinforcement corrosion.

3 Experimental work

3.1 Description of the Krk Bridge structure

The Krk Bridge connects the mainland and the island of Krk passing over the small island Sv.Marko, and consists of two reinforced concrete arches including the second largest conventionally reinforced concrete arch span in the world (390 m) (http://en.wikipedia.org/wiki/List_of_the_largest_arch_bridges). Total length of the bridge is 1310 m, which includes 96 m of road over the island Sv.Marko, Fig. 1. The bridge was constructed from 1976 until 1980.

During the more than 25 years in service the bridge has been exposed to strong winds (gale and sirocco), seawater splash and extensive wind blown chlorides all over the bridge substructure. High salinity of the water (3.5%) combined with the wind and the high temperature levels during summer periods accelerated the penetration of chlorides into the concrete, columns and arches, initiating corrosion of the reinforcement followed by microcracking and spalling of the concrete cover, Fig. 2. Therefore, structures of both arches are being repaired intensively during last 10 years [5]. Currently the structure of the small arch is being rehabilitated.

The original concrete composition is given in Table 1, and concrete properties in fresh and hardened state are given in Table 2 [6, 7].

Fig. 1 Krk bridge (view from the island towards mainland)

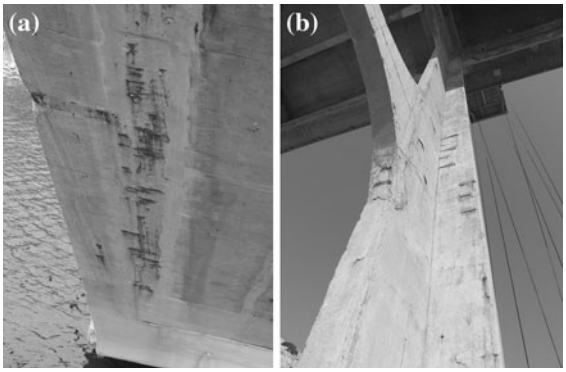

Fig. 2 Corrosion damage on **a** arch and **b** column

Table 1 Concrete mix design

Material	Comment	kg per 1 m^3
Cement ($w/c = 0.36$)	Blast furnace slag cement with 20% of slag (CEM II/A-S 42,5)	450
Water	Potable water	162
Aggregate (0–16 mm)	Alluvial crashed carbonate gravel	1869
Air-entrainer (Pumpcrete N)	0.15% by weight of cement	0.667
Superplasticizer (Fluidal VX-OC)	0.20% by weight of cement	0.890

Design values of concrete cover were 25 mm for columns and main span structures, and 30 mm for foundation elements, while in situ measured values of concrete cover were between 25 and 50 mm, with the mean value of 38 mm and standard deviation of 10 mm [8].

3.2 Chloride determination

Chloride contents (total amount of chlorides) were determined experimentally by standard laboratory method (potentiometric titration, according the national standard HRN B.C8.020). Concrete powder was sampled on site or in the laboratory from concrete cores.

The chloride content depends on the mass of concrete sample. To get a representative sample for the chlorides in the cement paste in concrete with a maximum grain size of 10–25 mm, 10–20 g of concrete powder is needed. Samples were taken from horizontal and vertical surfaces on the structure by ø18-mm drill, always getting the samples from minimum three holes at each location, or from slices cut from the cores and pulverized for chloride determination.

Positions of concrete samples for chloride determination, taken from the big arch structure are shown in Fig. 3 [8]. During the condition survey more than 1200

Table 2 Concrete properties

Property	Standard (today's equivalent)	Unit	Result
Fresh concrete			
Slump	EN 12350-2	cm	2.5
Bleeding	EN 480-4	ml	73.5
Density	EN 12350-6	kg/m^3	2520
Air content	EN 12350-7	%	2.6
Hardened concrete			
Compressive strength	EN 12390-3	N/mm^2	48.8–55.6
Flexural strength	EN 12390-5	N/mm^2	9.3
Static modulus of elasticity (stresses until 1/3 of compressive strength)	HRN U.M1.025	N/mm^2	41,300
Water permeability	EN 12390-8	mm	<10
Capillary absorption	EN 13057	$kg/(mm^2\ h^{0.5})$	0.63
Gas permeability	EN 993-4	$(\times 10^{-16})\ m^2$	1.66
Freeze-thaw resistance of concrete—internal structural damage	CEN/TR 15177	%	4

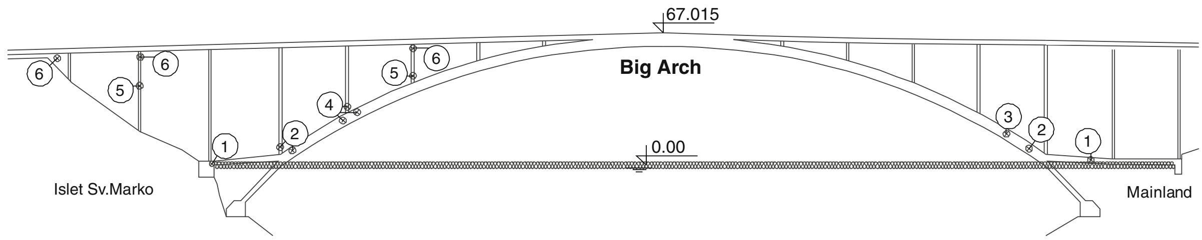

Fig. 3 Positions of taking samples for determination of chloride profiles

Table 3 Number and distribution of chloride profiles

Zone	m a.s.l.	Support	Columns	Arch	Total number of profiles per zone
1	0–10	–	10	30	40
2	10	–	20	60	80
3	20	–	–	40	40
4	30	–	20	20	40
5	40	–	32	16	48
6	62	8	32	16	56
Total number of profiles per element		8	114	162	284

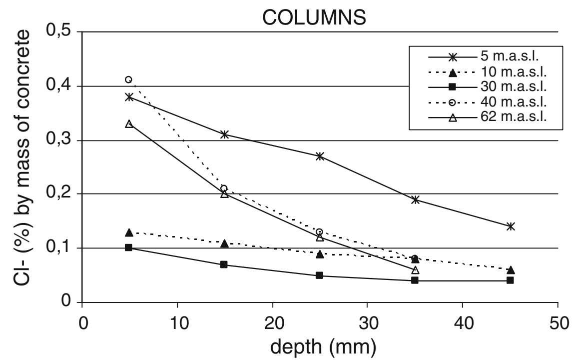

Fig. 5 Typical chloride profiles for columns

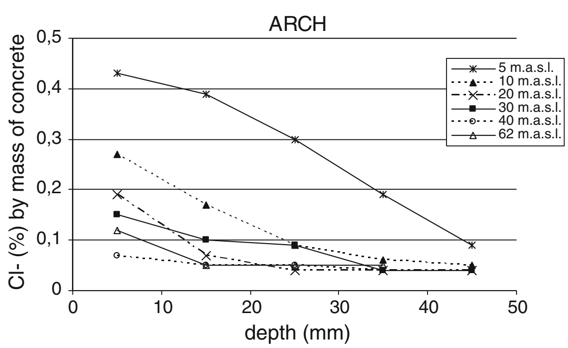

Fig. 4 Typical chloride profiles for arch

Table 4 Risk of corrosion initiation depending on the total chloride content [1]

Cl^- % by mass of cement	Cl^- % by mass of concrete for 450 kg cement/m^3	Risk of corrosion
>2.0	>0.37	Certain
1.0–2.0	0.19–0.37	Probable
0.4–1.0	0.08–0.19	Possible
<0.4	<0.08	Negligible

chloride analysis at 284 locations were performed, given in Table 3. In Figs. 4 and 5 averaged chloride profiles are presented for arch and columns, depending on the position of concrete sampling and height above sea level (a.s.l.), as shown in Table 3 and Fig. 3.

3.3 Threshold value for initiation of corrosion

It is well-known that the critical chloride concentration triggering reinforcement corrosion in concrete is a very uncertain value. This so-called threshold value depends on many factors, such as materials parameters, different exposure conditions, shape factors etc. When the threshold chloride concentration is achieved, the corrosion progress depends on the resistivity of the concrete, the temperature level and the availability of oxygen. The chlorides are partly bound chemically and physically, and partly free being available as catalysts for the electrochemical reaction with the steel [9, 10].

For structures exposed to very aggressive environment, such as at the Adriatic coast, the levels reported by Browne et al. [11] (Table 4) correspond well with such environment and are therefore often used for this purpose.

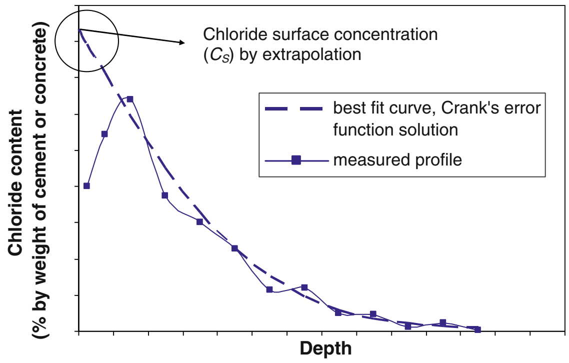

Fig. 6 Best fit curve based on Crank's error function solution to Fick's second law of diffusion

In this investigation of the Krk Bridge a total chloride threshold value of 0.60% by mass of cement, corresponding to 0.11% by mass of concrete at the reinforcement level, has been adopted. This rather tolerant value seems to comply reasonably well with the observations.

3.4 Analysis of chloride surface concentration

The measured chloride profiles have been analysed to derive the chloride surface concentration (C_S values) and these computed data have been used as a basis for the further analysis. The surface chloride concentration is determined by adjusting representative chloride profiles along with the Crank's solution of Fick's second law of diffusion using error function and then extrapolating the curve to the surface, as shown in Fig. 6 [12, 13]. This value represents then the calculated surface concentration, C_S, used further in the analysis.

In Fig. 7 the surface chloride concentrations are presented in relation to height above sea level (a.s.l.) and to the side of the structural element. High surface concentration at the level of 40–60 m above the sea level on north and east side of columns is explained by very strong wind, called Bora, blowing from northeast. Bora is a vigorous flow, a type of severe downslope windstorm, which varies in space and time and the location of occurrence. In the area of Krk Bridge related hourly mean wind speeds surpassing 20 m/s, with gusts reaching up to 50 or even 70 m/s, are common (hurricane speeds) [14, 15].

Due to the big differences in environmental load on different parts of the bridge elements caused by the microclimatic conditions, as shown in Fig. 7, it is difficult to define representative statistical value as critical chloride surface concentration. Therefore a nominal value for the design surface chloride concentration is accepted, based on EN 1990 Eurocode 0 [16, 17]. In Table 5 results of the statistical analysis of chloride surface measurements are presented, where design value of surface chloride concentration, C_{Sd}, is defined as:

$$C_{Sd} = C_{S\,mean} + 1.3\,\sigma_s \tag{1}$$

where $C_{S\,mean}$ is the mean value and σ_s the standard deviation. Coefficient 1.3 is accepted from [17] and means that 10% of the population has higher concentration than C_{Sd}.

3.5 Chloride diffusion coefficient

After determining the chloride profiles, the chloride diffusion coefficient was calculated for the representative chloride profiles by curve fitting along with the defined solution of Fick's second law by means of inverse error function [12]:

$$C(x,t) = C_i + (C_s - C_i)\cdot \mathrm{erfc}\,\frac{x}{\sqrt{4\cdot t\cdot D_{app}}} \tag{2}$$

where $C(x,t)$ is the chloride concentration at the depth x and at the time t, C_s calculated chloride surface concentration, C_i initial chloride concentration in the concrete mix, D_{app} apparent chloride diffusion coefficient, x depth from surface exposed to chloride attack, t time of exposure, and erfc is the complementary error function. Summary of the results of D_{Cl} calculation are presented in Table 6.

Statistical analysis of D_{app} gave the following results:

- average value for all zones: $D_{app} = 0.85 \times 10^{-12}$ m²/s,
- with the standard deviation: $\sigma_D = 0.51 \times 10^{-12}$ m²/s.

Results of the calculation of the apparent diffusion coefficient are in very good correlation with values from the literature. Since the concrete quality was from class 50, with w/c ratio of 0.36 and with 20% of slag, and that measurements were performed after 25 years of exposure, this value of the chloride diffusion coefficient is considered very realistic [18–21].

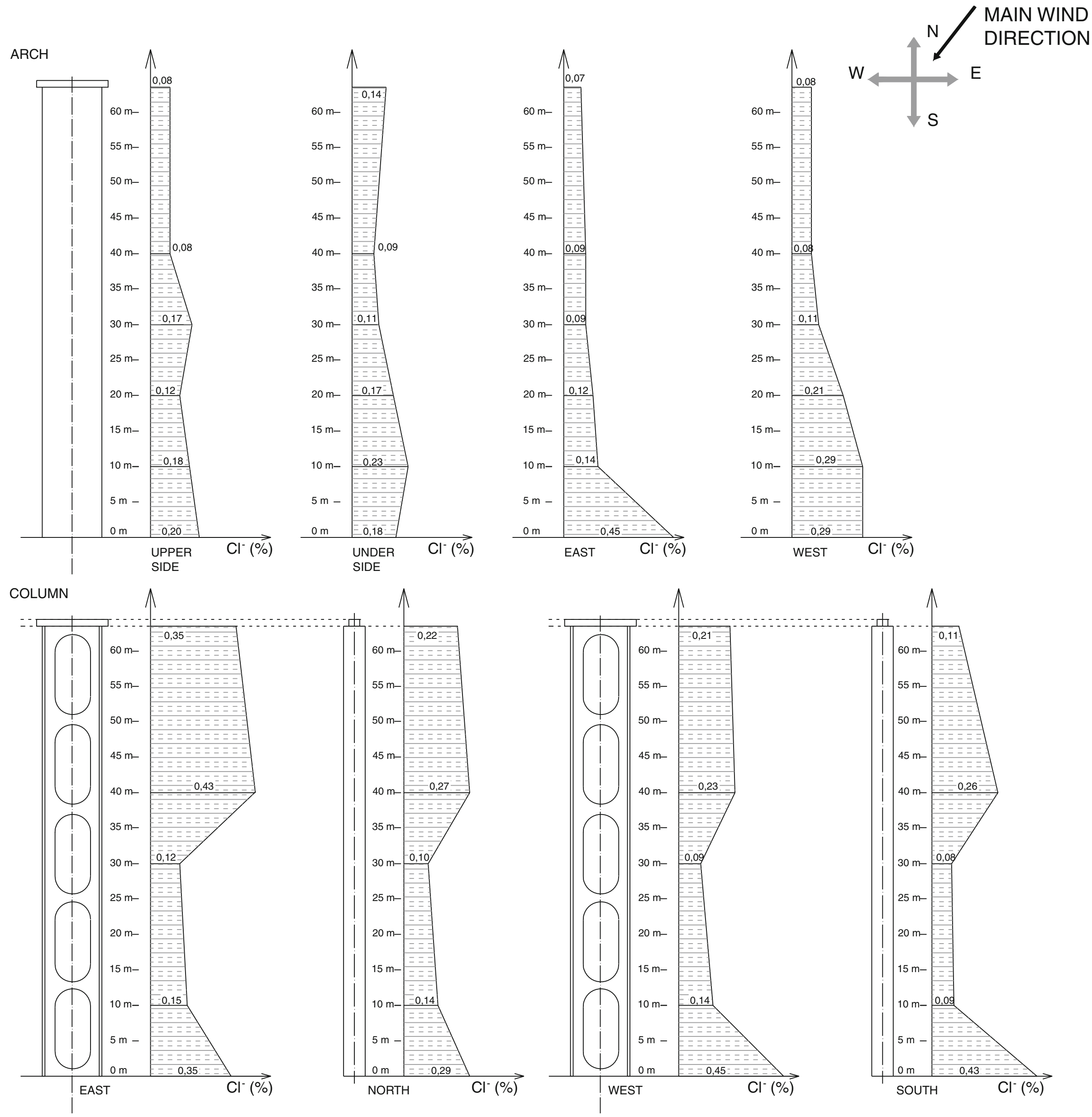

Fig. 7 Chloride surface concentration (Cl^- by mass of concrete) depending on the height above sea level and on the side of the structural element, arch and column

4 Applications of mathematical models

Taking into account the assumption that the diffusion is the predominant process of chlorides penetrating into the concrete, it is possible to describe mathematically the initiation period by Fick's second law of diffusion. This is the approach adopted by the three models used in this research, *Life-365*, *CHLODIF* and *DuraCrete*.

Within the models *Life-365* and *CHLODIF* the initiation period t_i and chloride profiles after certain time of exposure can be predicted by using similar assumptions. These models are deterministic and input parameters for both of them are describing

Table 5 Calculated surface chloride concentrations (% by mass of concrete) for columns and arch depending on the exposure zones

Exposure zone	Height (m a.s.l.)	Columns			Arch		
		$C_{S\ mean}$ (%)	σ_s (%)	C_{Sd} (%)	$C_{S\ mean}$ (%)	σ_s (%)	C_{Sd} (%)
1	0	0.38	0.07	0.47	0.31	0.14	0.49
2	10	0.13	0.04	0.18	0.21	0.06	0.29
3	20	0.11[a]	0.03[a]	0.15[a]	0.16	0.04	0.21
4	30	0.10	0.02	0.13	0.12	0.03	0.16
5	40	0.30	0.09	0.42	0.09	0.01	0.10
6	62	0.22	0.10	0.35	0.09	0.03	0.13

[a] Computed as an average value between zone 2 and zone 4

Table 6 Surface chloride concentrations and apparent chloride diffusion coefficients

Zone	Height (m a.s.l.)	C_{Sd} (% by mass of concrete)		D_{app} ($\times 10^{-12}$ m^2/s)	
		Columns	Arch	Columns	Arch
1	0	0.47	0.49	1.45	0.52
2	10	0.18	0.29	0.76	0.55
3	20	0.15	0.21	0.75	0.51
4	30	0.13	0.16	0.97	0.78
5	40	0.42	0.10	0.52	2.23
6	62	0.35	0.13	0.56	0.56

materials characteristics, structure details and environment conditions [2, 22–24].

The *DuraCrete* model, based also on error function solution of Fick's second law of diffusion, calculates the probability of corrosion initiation in relation to the time of exposure. By defining the acceptance criteria it is possible to predict structural performance for a pre-determined design service life with a selected level of reliability [2].

The time to initiation (t_i) may be calculated as the time necessary for the concentration of chloride ions to reach the critical value C_{cr} at the level of reinforcement, which is a function of the transport properties of concrete (usually the apparent diffusion coefficient), the surface chloride concentration defined by the environment, the thickness of the concrete cover and the chloride threshold value. The predicted initiation time will be taken as the design service life, which is justified by being on a safety side, since during this period no serious damage or consequences are assumed to develop, and there is always a "spare" time (propagation period), during which the structure can be repaired.

Extension of the service life can be achieved by regular interventions, i.e. by intensified maintenance and repair measures, when a structural damage is threatening. Maintenance of structures is necessary to ensure that a required performance is maintained above the critical level.

4.1 Service life prediction by *Life-365*

Life-365 is based on a 1-D and 2-D finite difference implementation of Fick's Second Law, the general advection–dispersion equation. 1-D solutions are provided by direct solution of the stiffness matrix, whereas 2-D solutions are estimated by a successive over-relaxation iterative approach. The time step used in the temporal derivatives is dynamically increased during the analysis to decrease analysis times [24].

The theoretical value of the apparent chloride diffusion coefficient at 28 days maturity, D_{28}, is dependent on the water–cementitious material ratio (w/c) of the concrete and is described by the following relationship:

$$D_{28} = 1 \times 10^{(-12.06+2.40w/c)}\ \mathrm{m}^2/\mathrm{s} \tag{3}$$

The chloride diffusion coefficient is furthermore also a function of time and temperature:

$$D(t,T) = D_{\mathrm{ref}} \cdot \left(\frac{t_{\mathrm{ref}}}{t}\right)^m \cdot \exp\left[\frac{U}{R} \cdot \left(\frac{1}{T_{\mathrm{ref}}} - \frac{1}{T}\right)\right] \tag{4}$$

where $D(t,T)$ is the diffusion coefficient at time t and temperature T, D_{ref} diffusion coefficient at some reference time t_{ref} ($D_{\mathrm{ref}} = D_{28}$ in *Life-365*), M constant (depending on mix proportions), $m = 0.2 + 0.4$ (%FA/50 + %SG/70), FA = Fly ash; SG = slag (ground granulated blast furnace slag), U activation energy of the diffusion process (35,000 J/mol), R gas constant (8.314 J K^{-1} mol^{-1}),

Table 7 Input parameters for *Life-365*

Input parameter		Value
Surface chloride concentration	C_s (% by mass of concrete)	0.17 for atmospheric zone (XS 1 acc. [23])
		0.48 for splash zone (XS 3 acc. [23])
Chloride threshold value	C_{crit} (% by mass of concrete)	0.11
Concrete cover	$x_c \pm \sigma_x$ (mm)	38 ± 10
Water–cement ratio	w/c (–)	0.36
Apparent diffusion coefficient (experimentally determined)	$D_{app} \pm \sigma_D$ ($\times 10^{-12}$ m^2/s)	0.85 ± 0.51
Age factor	m (–)	0
Theoretically predicted diffusion coefficient by the model	D_{theor} ($\times 10^{-12}$ m^2/s)	6.37 (for 28 days)
Age factor	m (–)	0.31

Table 8 The average air temperatures for the location of Krk Bridge (°C)

I	II	III	IV	V	VI	VII	VIII	IX	X	XI	XII
5.0	5.9	8.5	12.3	16.4	19.9	22.8	22.2	18.4	14.3	9.5	6.6

Table 9 Calculation results of time to corrosion initiation with *Life-365*

Input parameters				Time to corrosion initiation (years) for					
				Atmospheric zone (XS1)			Splash zone (XS3)		
Diffusion coefficient		$\times 10^{-12}$ m^2/s	Age factor, m (–)	$x_{c.1}$ = 28 mm	$x_{c.2}$ = 38 mm	$x_{c.3}$ = 48 mm	$x_{c.1}$ = 28 mm	$x_{c.2}$ = 38 mm	$x_{c.3}$ = 48 mm
D_{app}	$D_{1,min}$	0.34	0	50.2	90.3	138.2	16.1	27.7	42.6
	$D_{2,average}$	0.85	0	21.8	37.8	57.1	8.0	12.7	18.7
	$D_{3,max}$	1.36	0	14.7	24.7	36.8	6.0	8.9	12.7
D_{theor}	$D_{4,theor}$	6.37	0.31	15.7	28.5	44.2	5.7	8.9	13.3

and T is the absolute temperature, $T_{ref} = 293$ K (20°C).

Based on the chloride surface concentrations two typical zones of exposure are recognized from Fig. 7, splash and atmospheric zones for both structural elements: columns and arch. Input parameters for Krk Bridge for the calculation of service life with *Life-365* are given in Table 7, describing column, as a critical case of the 2-D problem. In Table 8 are given average month temperatures in the area of Krk Bridge.

Results of calculations performed with model *Life-365* for each zone of exposure are presented in Table 9 and in Figs. 8 and 9.

In Fig. 8 chloride profiles obtained from the model *Life-365* are compared to the experimentally determined chloride profiles for different exposure zones and different concrete properties. It can be seen that model gives higher chloride concentrations comparing to the experimental values of chloride concentrations, but this is mainly due to the input of design value of chloride surface concentration, which includes dissipation of results and safety coefficient, as described in Sect. 3.4. When the chloride surface concentration is set to the lower value (really measured), profiles are fitting very well [1].

The results of time to corrosion initiation, t_i, calculation based on theoretical input parameters

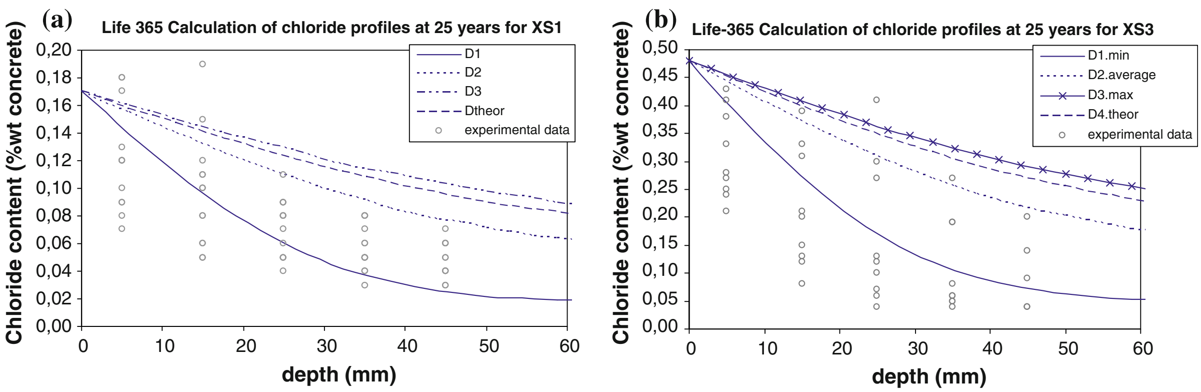

Fig. 8 Calculated chloride profiles with *Life-365* for 25 years of exposure in relation to different material parameters for **a** atmospheric zone and **b** splash zone and compared with experimental data

(diffusion coefficient and age factor), are very close to the results obtained from the worst case of experimental input—maximum value of the apparent diffusion coefficient, as it can be seen from Fig. 9a, b. In this way the results obtained from the model, based completely on the assumptions from model, are predicting the "real" worst case, and being on the side of a safer prediction of structural behaviour.

It can be seen that with a definition of three values of diffusion coefficient and three values of concrete cover, the range of time to corrosion initiation is rather large (see Table 9), but it can be very well used as a timetable for maintenance and repair actions during structural service life if interpreted by an expert. This is generally in very good correlation with the real situation of concrete structures such as Krk Bridge exposed to aggressive environment, where the rate of degradation processes and defects in concrete are greatly dependent on micro-conditions and built-in concrete properties, generating variety of structural performance by elements. Serious repair works of columns and head beams on big arch of Krk Bridge started already in 1987, only 7 years after its finish. In 1988 started repair and protection of lower parts of arches and supports, and until today Krk Bridge as such is under continuous repair process [5].

4.2 Service life prediction by CHLODIF

The process of a continuous diffusion process of chloride ions into a reinforced concrete structure with a time-varying diffusion coefficient and chloride surface concentration is described by the following equation, according to [22, 23, 25]:

for $0 \le C_0 \le C_{\max}$

$$C(x,t) = [C_0 + k(t-1)]\left(1 - \operatorname{erf}\frac{x}{2\sqrt{\tau}}\right) + k\left[\left(1 + \frac{x^2}{2\tau_1}\right)\left(1 - \operatorname{erf}\frac{x}{2\sqrt{\tau_1}}\right) - \frac{x}{\sqrt{\pi\tau_1}}e^{-\frac{x^2}{4\tau_1}}\right], \tag{5}$$

until reaching the maximum surface concentration $C_0 = C_{\max}$ when the following solution becomes valid:

$$C(x,t) = C_0\left(1 - \operatorname{erf}\frac{x}{2\sqrt{\tau}}\right), \quad C_0 = C_{\max} \tag{6}$$

where C_0 is the chloride concentration at $x = 0$, $C_{\max}$ maximum concentration of chloride ions at $x = 0$, τ the substitution by which variation of $D_{\mathrm{Cl}}-$ with time is taken into account [24] and

$$d = D(t)\mathrm{d}t \quad \text{thus} \quad \tau = \int_0^t D(s)\mathrm{d}s \tag{7}$$

k is the coefficient of linear increase of initial concentration [26]

$$D_{\mathrm{Cl}^-} = D_{w/c} \times D_0 \times t^{-0.1} \tag{8}$$

where $D_{w/c}$ is the chloride diffusion coefficient depending on *w*/*c* ratio (cm^2/s), D_0 coefficient which takes into account the type of cement, and *t* is the age of the structure (years).

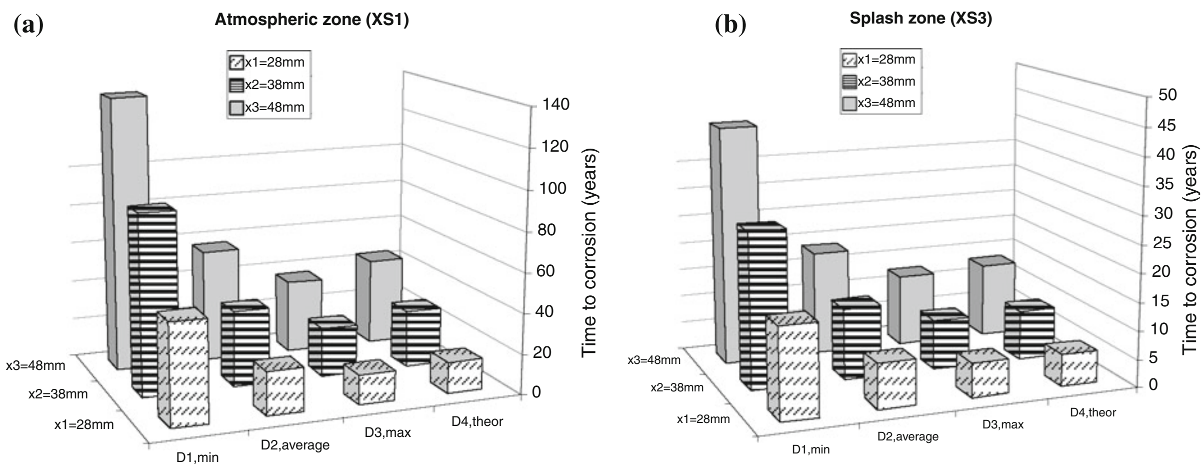

Fig. 9 Time to corrosion initiation in relation to different material parameters and concrete cover deviations for **a** atmospheric zone (XS1) and **b** splash zone (XS3)

Input parameters for the calculation of the service life of Krk Bridge are given in Table 10. Due to the assumptions of model, it was not possible to set the age factor equal to zero, since it is fixed value, set in the calculations to 0.10.

The results of calculations with *CHLODIF* for each zone of exposure are presented in Table 11 and in Figs. 10 and 11.

Calculated chloride profiles with the model *CHLODIF* for different exposure zones and different material properties are compared to the experimentally determined chloride profiles in Fig. 10. It can be seen that model gives very close or lower values of chloride concentrations comparing to the experimental ones in the atmospheric zone, while in the splash zone calculated chloride profiles are mainly close or above the experimental ones.

Results of time to corrosion initiation for both zones of exposure are unrealistically high, especially for the atmospheric one, comparing to the actual state of Krk Bridge. Multiplying correction factor for diffusion coefficient based on cement type (0.30 for slag cement) and aging factor (0.10) obviously gives unrealistically good results.

4.3 Service life prediction by DuraCrete

As for the other models corrosion is initiated when the chloride content around the reinforcement exceeds a critical threshold value. This state is defined as the design service life representing a serviceability limit state (SLS). Assuming that the initial chloride content of the concrete is zero, the design equation for this SLS is given by [2, 20]:

$$g = c^{\mathrm{d}}_{\mathrm{cr}} - c^{\mathrm{d}}(x,t) = c^{\mathrm{d}}_{\mathrm{cr}} - c^{\mathrm{d}}_{\mathrm{s,cl}}\left[1 - \mathrm{erf}\left(\frac{x^{\mathrm{d}}}{2\sqrt{\frac{t}{R^{\mathrm{d}}_{\mathrm{cl}}(t)}}}\right)\right] \tag{9}$$

where $c^{\mathrm{d}}_{\mathrm{cr}}$ is the design value of the critical chloride concentration (% Cl$^-$/binder), $c^{\mathrm{d}}_{\mathrm{s,cl}}$ design value of the chloride surface concentration (% Cl$^-$/binder), x^{d} design value of the cover thickness (mm), $R^{\mathrm{d}}_{\mathrm{cl}}$ design value of the chloride resistance (year/mm^2), t time (year), and erf is the error function. In this equation $g = 0$ denotes the point in time of corrosion initiation.

The probability of corrosion initiation within the period of time [0; T], $P_{\mathrm{f}}(T)$ is defined as

$$P_{\mathrm{f}}(T) = 1 - P\{g(\mathbf{x},t) > 0 \quad \text{for all } t \in [0;T]\}. \tag{10}$$

The acceptance criterion is given in terms of a reliability index, β, defined by

$$\beta = -\Phi^{-1}(P_{\mathrm{f}}) \tag{11}$$

where P_{f} is the probability of corrosion initiation occurring within the considered reference period (design service life). In the present case it has been accepted that the probability of corrosion initiation due to chlorides may be set as high as 10^{-1},

Table 10 Input parameters for *CHLODIF*

Input parameter		Value
Surface chloride concentration	C_s (% by mass of concrete)	0.17 for atmospheric zone (XS 1 acc. [23])
		0.48 for splash zone (XS 3 acc. [23])
Chloride threshold value	C_{crit} (% by mass of concrete)	0.11
Concrete cover	$x_c \pm \sigma_x$ (mm)	38 ± 10
Water–cement ratio	w/c (–)	0.36
Correction factor for diffusion coefficient for slag cement	D_0	0.30
Apparent diffusion coefficient (experimentally determined)	$D_{app} \pm \sigma_D$ ($\times 10^{-12}$ m²/s)	0.85 ± 0.51
Age factor	m (–)	0.10
Theoretically predicted diffusion coefficient by the model	D_{theor} ($\times 10^{-12}$ m²/s)	0.94 (for 365 days)
Age factor	m (–)	0.10

Table 11 Calculation results of time to corrosion initiation with *CHLODIF*

Input parameters				Time to corrosion initiation (years) for					
				Atmospheric zone (XS1)			Splash zone (XS3)		
Diffusion coefficient		$\times 10^{-12}$ m²/s	Age factor, m (–)	$x_{c.1}$ = 28 mm	$x_{c.2}$ = 38 mm	$x_{c.3}$ = 48 mm	$x_{c.1}$ = 28 mm	$x_{c.2}$ = 38 mm	$x_{c.3}$ = 48 mm
D_{app}	$D_{1,min}$	0.34	0.10	>200	>200	>200	40	78	130
	$D_{2,average}$	0.85	0.10	138	>200	>200	15	28	47
	$D_{3,max}$	1.36	0.10	82	161	>200	9	17	28
D_{theor}	$D_{4,theor}$	0.94	0.10	122	>200	>200	13	25	42

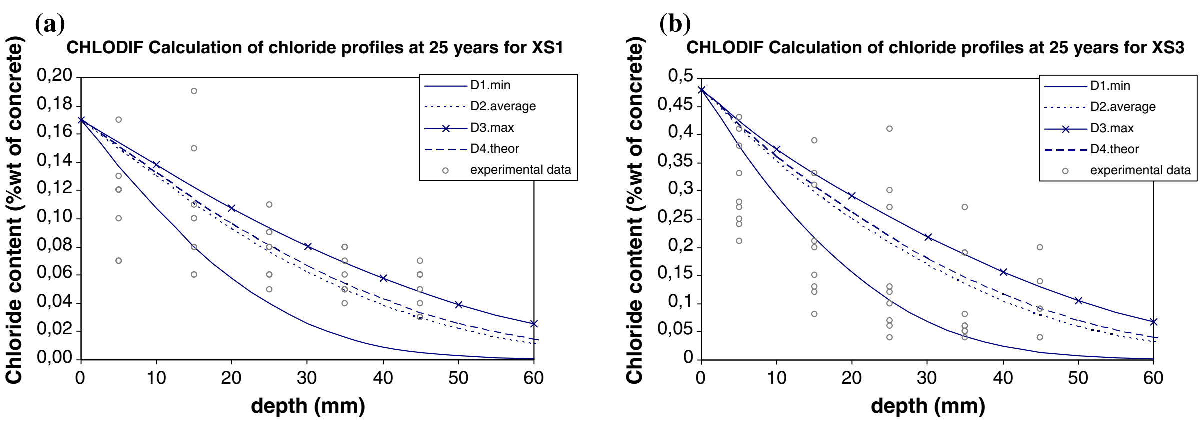

Fig. 10 Calculated chloride profiles with *CHLODIF* for 25 years of exposure in relation to different material parameters for **a** atmospheric zone and **b** splash zone and compared with experimental data

corresponding to a reliability index β of approximately 1.3.

For the probabilistic performance-based service life design according *DuraCrete* two main sets of input parameters were used: (i) the theoretical diffusion coefficient with the age factor different from zero following the recommended values given in the original DuraCrete Report [2], and (ii) the experimental (apparent) diffusion coefficient of an older existing structure with the age factor equal to

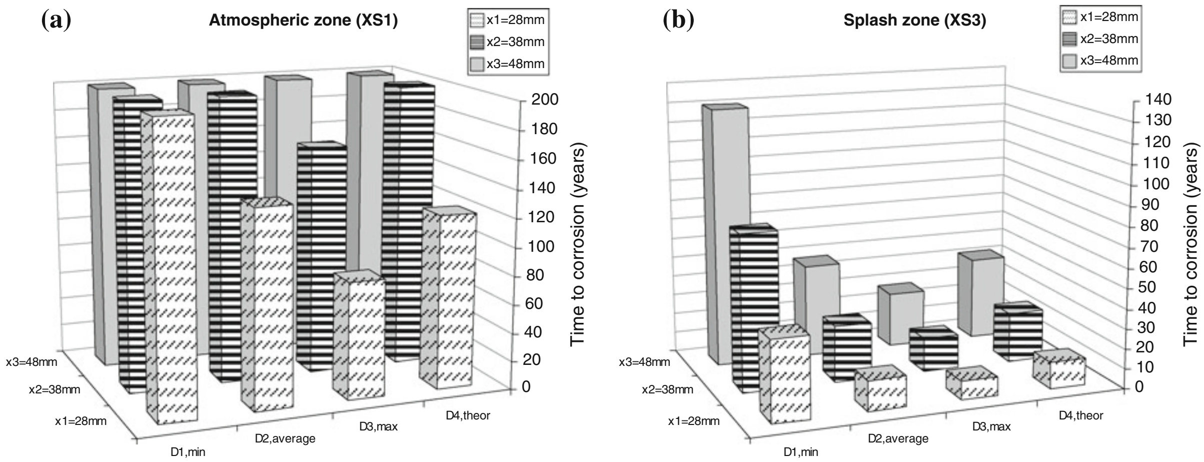

Fig. 11 Time to corrosion initiation in relation to different material parameters and concrete cover deviations for **a** atmospheric zone (XS1) and **b** splash zone (XS3)

zero. The theoretical value of the diffusion coefficient was calculated based on concrete mixture and with 3 empirical models, according to database at [27–29].

Input parameters and all chosen coefficients and distribution functions are given in Table 12.

The limit state equation based on Fick's second law of diffusion (Eq. 9) is modelled in computer program COMREL, which is a sub-program of STRUREL. Shortened outcome of the calculation for Krk Bridge is presented in Fig. 12.

Based on the input parameters for Krk Bridge the results from the *DuraCrete* calculations are the following:

- The theoretical values of input parameters showed greater deviations in the *DuraCrete* model for the concrete types of good quality. Specifically, the predicted reliability of the structure was higher than that actually found. This was clearly due to the unrealistic prediction of a large reduction in diffusion coefficient with time for the type of cement used, which clearly does not correspond to the real in-field situation. The theoretical value of the age factor is 0.65 and 0.85 respectively. It is, therefore, suggested that this parameter should be further investigated and compared with the actual condition of structures in operation after a longer period of time. Experimental input parameters correspond very well to the results of calculation when the age factor n_{Cl} was set to the values of 0.20 and 0.25.
- No probability of corrosion in the atmospheric zone at an age of 25 years.
- In the splash zone after 10 years of exposure reliability index equals to 1.30 for the experimental input parameters.

5 Discussion of the results

When chloride penetration into concrete should be predicted, the key parameters are a diffusion coefficient and its change with the time and environmental load defined as chloride surface concentration. The input parameters used for the predictive calculations were experimental data representing the existing Krk Bridge structure and theoretically assumed values of input parameters, representing the recommended values from the manuals of the applied models.

The diffusion coefficient is a function of many factors, but mainly is theoretically predicted on the basis of water–cement ratio, cement type and cement additives. Time-dependent changes of diffusion coefficient are usually described by power law, and it decreases with time, due to the continuous hydration and binder reactions which densify the concrete with time [30, 31]. The time-dependency is varying very much for different types of concrete, especially for different binders, and is extremely influencing results of prediction. This is also supported by this study, where age factors were between 0.10 (*Chlodif*) and 0.85 (*DuraCrete*), given in Tables 7, 10 and 12. The

Table 12 Input parameters for *DuraCrete* calculation

Input parameter			Atmospheric zone (XS1)		Splashing zone (XS3)	
Parameter	Unit	Distribution function	Mean value	SD	Mean value	SD
C_S	% by mass of concrete	ND	0.17	0.03	0.48	0.15
C_{cr}	% by mass of concrete	LN	0.11	0.03	0.11	0.03
x_c	mm	LN	38	10	38	10
$k_{c,cl}$	–	–	1	–	1	–
$k_{e,cl}$	–	Gamma	0.78	0.10	0.78	0.10
D_{Cl^-theor}	$\times 10^{-12}$ m^2/s	ND	2.84	0.35	2.84	0.35
Age factor n_{Cl}	–	LN	0.85	0.10	0.65	0.10
D_{Cl^-exp}	$\times 10^{-12}$ m^2/s	ND	0.85	0.51	0.85	0.51
Age factor n_{Cl}	–	–	0	–	0	–

SD standard deviation, *ND* normal (Gauss) distribution

LN, lognormal distribution; D_{Cl^-theor}, theoretical value of diffusion coefficient, with age factor $\neq 0$; D_{Cl^-exp}, experimental value of diffusion coefficient, with age factor $= 0$; $k_{c,cl}$, curing factor; $k_{e,cl}$, environment factor

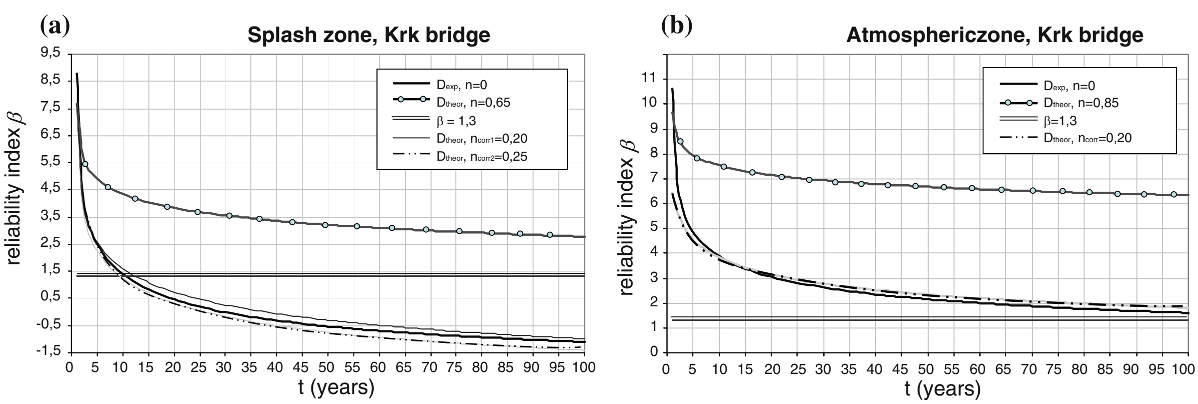

Fig. 12 Reliability index β change within the time for the **a** splash zone and **b** atmospheric zone

best results gave *Life-365* where the age factor was 0.31. When the age factor in *DuraCrete* was set to the values of 0.20 and 0.25 results of calculation analysis corresponded very well to the experimental ones. This brings the conclusion that time dependent change of diffusion coefficient of concrete with 20% of slag addition to cement is best described with the value of age factor from 0.20 to 0.30.

Dependences of calculation results from *Life-365* and *Chlodif* on diffusion coefficient and concrete cover depth are shown in Fig. 13a, b. It is obvious that *CHLODIF* gives greater differences in results and unrealistically large range of time. *Life-365* gives reasonable results, which are in good correlation with the real situation of Krk.

Chloride surface concentration is in this study used on the basis of experimental values, and its evolution is not discussed deeper in this study, but it is from a great importance to predict real value of surface concentration which is according to [32] the most crucial parameter in the prediction.

When calculations are made with the apparent diffusion coefficient defined by minimum, mean and maximum tested values, the result is provided as boundary curves for chloride profiles after a certain time. This result correlates very well with the real situation where chloride profiles have great dissipation of results. This method can be used during design stage, from three chloride profiles obtained from three diffusion coefficients, defined as average value

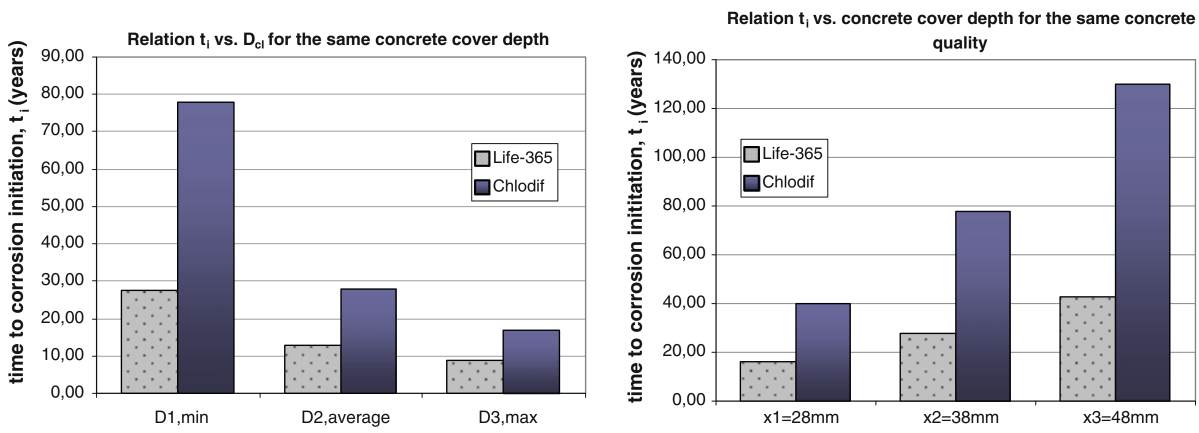

Fig. 13 Comparison of results from *Life-365* and *CHLODIF*: **a** dependence on concrete cover depth, **b** dependence on water/cement ratio

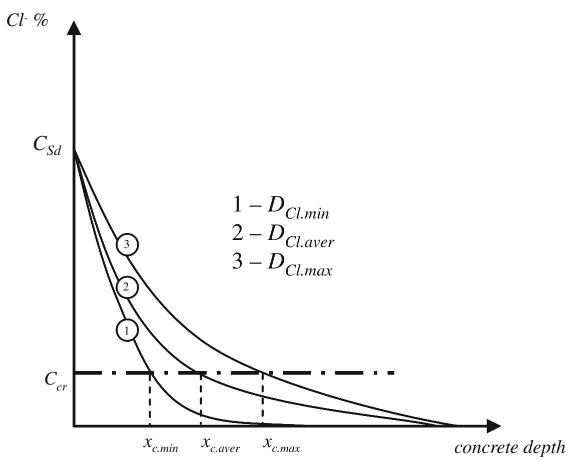

Fig. 14 Determination of design values of concrete cover depth on the basis of $D_{\mathrm{cl.aver}} \pm \sigma_{\mathrm{D}}$

and average value plus and minus the standard deviation, and with defined critical chloride value, it is possible to determine minimum, average and maximum concrete cover, as shown in Fig. 14. In this way simple deterministic models could be used very well for better service life prediction.

From the available results using *DuraCrete* there is obviously a very good correlation when using the experimental input values. But with the theoretical input values (diffusion coefficient and its time change, i.e. the age factor) *DuraCrete* is giving too high reliability for the concrete of better quality class (case of Krk bridge), comparing to the real state of the structure.

From the *DuraCrete* calculation (Fig. 15) for this period of exposure the β value is 0.23 (dashed arrows), which corresponds to 40% probability of corrosion occurrence. From Fig. 15 it can be also seen that the limit reliability index of 1.30 is attained after 10 years of exploitation (lined arrows), which is in a good correlation to the results from deterministic model.

6 Conclusions

(1) Service life design models should be calibrated against empirical data in order to determine their predictions availability. In the case when experimental input parameters were used in calculations, the results obtained using mathematical models showed much better match with the real state after 25 years in operation for all the zones of the environmental influences.

(2) The theoretical values of input parameters showed greater deviations in the *DuraCrete* model for the concrete types of good quality. Specifically, the predicted reliability of the structure was higher than that actually found. This was due to the prediction of a large reduction in diffusion coefficient with time, which clearly does not correspond to the real in-field situation. It is, therefore, suggested that this parameter should be further investigated and compared with the actual condition of the structures in operation after a longer period of

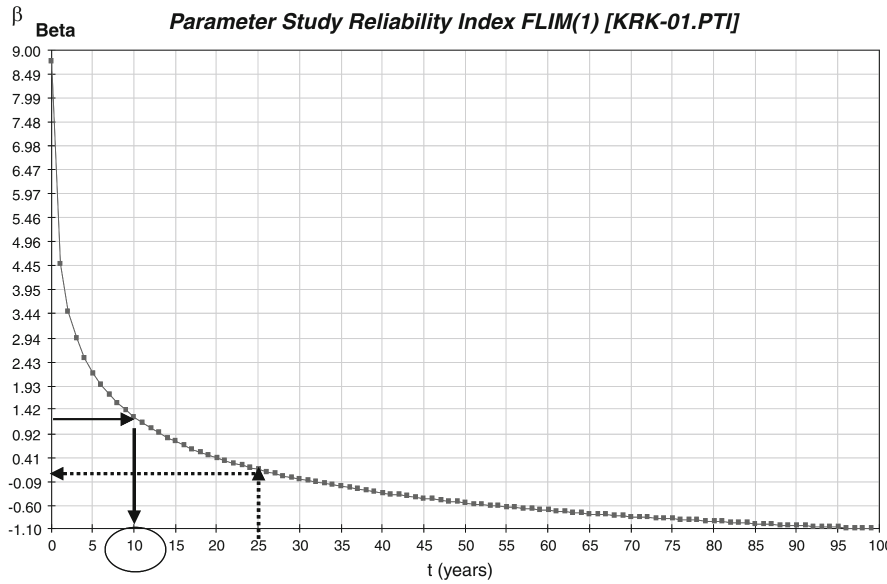

Fig. 15 Reliabiliy index for Krk bridge calculated with *DuraCrete* model

time. The observations confirm that ageing factors in the range of 0.2–0.3 for the type of cement and concrete mix in question are more realistic than the originally proposed values of 0.65–0.85. On the basis of this case-study, it can be concluded that the probabilistic approach to durability design of structures, *DuraCrete*, is reasonable when updated materials parameters are used.

(3) When calculations are made with deterministic models based on the apparent diffusion coefficient defined by minimum, mean and maximum tested values, the result is provided as boundary curves for chloride profiles after a certain time. This result correlates very well with the real situation of the structures, and can be used during design stage, when from three chloride profiles it is possible to determine minimum, average and maximum concrete cover needed. Results of deterministic model *Life-365* correspond very well to real condition of the analysed structure of Krk Bridge. Service life model *Chlodif* is too sensitive, as a consequence of multiplying correction factors for cement type and age factor, which gives unrealistically large range of time to corrosion initiation.

(4) In this case study high concentrations of chlorides on the concrete surface were found 40 and 60 m above sea level at northern and eastern column side, which is mainly influenced by strong wind blowing from N and NE at this location. This indicates that it is from a great importance to collect data from the real structures such as environmental conditions (wind, temperature, humidity etc.) which are influencing formation of chloride surface concentration, one of the most crucial parameters in the service life prediction.

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