Probabilistic assessment of concrete structure durability under reinforcement corrosion attack

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Abstract: In the context of performance-based approaches, sustainability and whole life costing, the concrete structure durability issue has recently gained considerable attention. The present paper deals with service life assessment utilizing Durability Limit States specialized for concrete structures. Both initiation and propagation periods of reinforcement corrosion are considered and a comprehensive choice of limit states is provided. The approach is based on degradation modelling and probabilistic assessment, enabling the evaluation of service life and the relevant reliability level, serving thus to facilitate the effective decision making of designers and clients. For this purpose the selected analytical models for degradation assessment are randomized and appropriate software has been developed. Three numerical examples are presented: a comparison of modelled carbonation depth with in-situ measurements on a cooling tower, and analyses of crack initiation due to corrosion and loss of reinforcement cross section.

Keywords: reliability, durability, service life, modelling, concrete, reinforcement corrosion

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Introduction

During the last twenty years the concrete structure service life issue has been given considerable attention and a great number of works have been published. As it is not in the scope of this paper to review them, let us mention only a few: Siemes et al. (1985), Sarja and Vesikari (1996), DuraCrete (1999), Rostam (2005).

In the context of performance-based approaches, sustainability and whole life costing, time is the decisive variable and the durability issues are pronounced – often within the framework of *Performance-Based Design*, the advanced trend in structural engineering design. This approach deals with *durability* and *reliability* issues, which both rank amongst the most decisive structural performance characteristics. This is clearly demonstrated by numerous international events and is reflected also in recent standardization activities: the ISO 13823 (2008) and *fib*-Model Code (2010). Both these documents advocate probabilistic approaches and enhance the *design of structures for durability* – i.e. a time-dependent limit state approach with service life consideration. The prescriptive approach of current standards does not directly allow for design focused on a specific (target) service life and/or a specific level of reliability. Advanced design for durability requires dealing with the inherent uncertainties in material, technological and environmental characteristics while assessing the service life of a structure. It appears that a more consistent approach to the durability design of concrete structures is needed, i.e. fully probabilistic durability design, which necessarily requires the utilization of stochastic approaches, analytical models of degradation effects based on experimental evidence and relevant observations of structures in real conditions, and also simulation techniques. The theoretical apparatus for this approach has already been developed but its utilization in practice is still rare and surprisingly even some recent research works are based on deterministic approaches only – see e.g. (Wang and Liu 2009).

Broad application of probabilistic design is still prevented by the insufficient dissemination of basic ideas or by the lack of efficient and user friendly design instruments (software and other).

Reinforced concrete is supposed to be very durable and is a widely used construction material. Despite this fact there are still a large number of cases with severe degradation and costly reconstruction work needed due to reinforcement corrosion. When considering the Limit States (LS) caused by the degradation of reinforced concrete structures, four kinds of attack may be distinguished:

(i) mechanical (mechanical load – static or dynamic),

(ii) chemical (carbonation, chloride and acid attack),

(iii) electrochemical (corrosion of reinforcement) and

(iv) physical (freeze-thaw, abrasion, fire and others).

One of the most frequent types of degradation of concrete structures is corrosion of reinforcement, which has significant (negative) implications for life cycle costing. Therefore, the present text focuses on cases (ii) and (iii). Owing to either carbonation of the concrete or the ingress of chlorides into the concrete, depassivation of reinforcing steel occurs (the initiation period); this may be followed by a steel corrosion process (the propagation period) – see e.g. (Tuutti 1982). With the focus on these periods, the modelling of relevant LS and degradation models is dealt with in the present paper. The (i) and (iv) types of concrete degradation are not encompassed in the present paper and are discussed elsewhere; e.g. frost or fire attack on concrete (*fib*-Model Code 2010, Matesová et al. 2006).

In the present paper the Durability Limit States (DLS) focused on the corrosion process in concrete structures are described. Some relevant analytical models (adopted from the literature) for reinforced concrete degradation assessment are randomized and used in a software tool developed by the team of authors: Novák et al. (2003) and Teplý et al. (2007)

allowing thus for the full probabilistic design/assessment of reinforced concrete structures in terms of degradation. The possibilities of the software are shown in numerical examples.

Design for durability

Let us note that the reliability approach (and hence the design process) has to cover, apart from safety and serviceability, also *durability* – see (EN 1990 2002). Durability is related to the *design working life* (or service life). The most generally used design method – the partial safety factor method – does not assess the reliability level directly, i.e. it does not arrive at a specified value of relevant reliability level and is not suitable for service life assessment. This situation can be amended by utilization of the full probabilistic approach, which is legally applicable (as an alternative to the partial safety coefficients approach) according to both the ISO and Eurocode documents, but not commonly accepted in practice.

The Ultimate Limit State (ULS) and the Serviceability Limit State (SLS) are assessed while designing a concrete structure. The general condition for the probability of failure P_f reads:

$$P_f = P(A \geq B) \leq P_d \tag{1}$$

where A is the action effect, B is the barrier and P_d is the design (acceptable, target) probability value. The index of reliability β is alternatively utilized instead of the probability of failure in practice. Generally, both A and B (and hence P_f or β) are time dependent; this has not been considered for common cases of ULS or SLS in design practice very frequently up to now.

Eq. (1) may be also expressed by means of service life format as:

$$P_f = P(t_S \leq t_D) \leq P_d \tag{2}$$

where t_D is design life and t_S can be determined as the sum of two service-life predictors:

$$t_S = t_i + t_p \tag{3}$$

where t_i is the time of the initiation of reinforcement corrosion and t_p is the service life after corrosion initiation (the propagation period).

Considering the durability issue, a new category of LS has been introduced which is intended to prevent the onset of deterioration or allow only a limited range of degradation. The new LS category represents such a type of limit states which may be called Durability Limit States (DLS) – see (ISO 13823 2008, *fib*-Model Code 2010, Sarja 2006). DLS may be formally regarded as belonging in the SLS category.

When considering the degradation of reinforced concrete structures, the corrosion of reinforcement is the dominating effect. In the context of the initiation period only one DLS can be recognized/defined – depassivation of reinforcement due to carbonation or chloride penetration creating the possible starting point for reinforcement corrosion. The relevant values of variables A and B used in Eq. (1), which are random quantities, have to be assessed by utilization of a suitable degradation model or by field or laboratory investigations. For the purposes of the former case effective probabilistic software tools are needed.

Initiation period

The degradation phenomena affecting RC structures dealt with in this paper are concrete carbonation, ingress of chloride ions and steel corrosion models.

The durability-oriented limit state condition (Eq. 1) specialized for the case of *carbonation* may be written as:

$$P_f(t_D) = P\left\{a - x_c\left(t_D\right) \le 0\right\} \le P_d \tag{4}$$

where a is concrete cover and x_c is the depth of carbonation at time t_D = design service life. Note that the carbonation process is driven by the diffusivity of ambient CO_2 in concrete and the reactivity of CO_2 with concrete. The CO_2 penetrating from the surface decreases pH to a value of 8.3. When the carbonation depth equals the concrete cover, the steel is depassivated and corrosion may start (when oxygen and moisture is present). The rate of carbonation progress from the concrete surface to the reinforcement depends on many parameters, e.g. concrete cover thickness and permeability, the ambient temperature, relative humidity and carbon dioxide content, whereas the concrete cover permeability itself depends on the concrete mix type and composition, the aggregate gradation and the processing and curing of the concrete mix.

Considering *chloride ingress* (e.g. due to de-icing salts), Eq. (1) may be transformed into the formula:

$$P_f(t_D) = P\left\{C_{cr} - C_a\left(t_D\right) \leq 0\right\} \leq P_d \tag{5}$$

where C_{cr} is the critical concentration of dissolved Cl^- leading to steel depassivation and C_a is the total concentration of Cl^- at the reinforcement at time t_D. Generally, this quantity is the sum of the initial concentration $C_{a,i}$ (due to chloride-contaminated compounds of concrete) and $C_{a,e}$ (the Cl^- concentration resulting from external sources, i.e. de-icing salts or sea water effects). Condition (5) is applicable for new as well as existing structures. Both limit conditions (Eqs. 4, 5) represent the initiation period limits and are – in the sense of degradation – conservative limits; they describe a situation where corrosion has not started yet. Typically, these conditions fall into the DLS category.

Details regarding the chloride diffusion process are described elsewhere, e.g. in (*fib*-Model Code 2010, Papadakis et al. 1996). From the steel corrosion point of view let us only mention that free chlorides (dissolved in pore solution) destroy the passive film of the steel rebar and lead to possible corrosion as they reduce the pH of the pore water, decrease the solubility of $Ca(OH)_2$ and increase the electrical conductivity and the moisture content due to the hygroscopic properties of chloride salts (Papadakis et al. 1996, Hunkeler et al. 2005).

The chloride threshold concentration may be presented by means of the total amount of chloride by weight of cement, the amount of free chloride, the concentration ratio of free chloride ions to hydroxyl ions or the ratio of acid-soluble chloride content and the acid neutralization capacity (the content of acid needed to reduce the pH of concrete and cement paste suspended in water to a particular value) (Ann and Song 2007). In terms of currently used representations, the total chloride content related to the cement weight is considered to be the best alternative. The value of 0.4 % for a building exposed to a European climate and 0.2 % for structures exposed to a more aggressive environment are currently suggested as the chloride threshold concentration (Glass and Buenfeld 1995, Duprat 2007). In the *fib*-Model Code (2010) the lower boundary of critical chloride content has been specified as 0.20 and the mean value as 0.60 [wt.-%/cement]; the statistical characteristics of $C_{cr.}$ can be quantified as follows: C_{cr}: *beta distribution* (m = 0.6; s = 0.15; a = 0.2; b = 2.0). According to EN 206-1 (2000), the maximal admissible value of the initial chloride content $C_{a,i}$ in concrete reinforced by steel rebars is 0.4 % (with respect to the weight of cement in the concrete mix), which is in reasonable correlation with the above-mentioned threshold concentration values.

Propagation period

Concentrating on reinforcement corrosion, the following LS may be specified:

(i) The rate of steel corrosion is governed by (among other factors) the availability of water and oxygen. Also, the presence of chlorides in the concrete surrounding the steel bars may influence the corrosion. The volume expansion of rust products develops tensile stresses in the surrounding concrete leading to *concrete cracking* (mainly affecting the concrete cover). The relevant limit condition for DLS may be constructed either with the tensile stress limit or crack width limit:

$$P_f(t_D) = P\left\{\sigma_{cr} - \sigma\left(t_D\right) \le 0\right\} \le P_d \quad (6)$$

$$P_f(t_D) = P\left\{w_{cr} - w_a\left(t_D\right) \le 0\right\} \le P_d \quad (7)$$

where σ_{cr} is the critical tensile stress that initiates a crack in concrete (on an interface with a reinforcing bar), σ is the tensile stress in concrete at the time of design service life t_D. The w_{cr} in (7) is the critical crack width on the concrete surface and w_a is the current crack width on the concrete surface at time t_D. Eq. (6) is typically a DLS, while Eq. (7) is either a DLS or an SLS depending on the w_{cr} value, which may have a considerable impact on durability. Note that the exceeding of w_{cr} or σ_{cr} depends not only on structural degradation caused by corrosion of reinforcement but also may co-act with a stress field developed due to other acting loads depending on the type of structure and its configuration. Also, note that the σ_{cr} and σ mentioned above should demonstrate the tensile strength of cementitious material surrounding the reinforcement. Due to the thinness of this layer and due to the fact that Eq. (6) should be viewed more generally in practical situations (considering also the possible combination with mechanical loading effects), the tensile stress of concrete is utilized. It is commonly treated in this way by other authors too; see e.g. (Liu and Weyers 1998, Li et al. 2006). Note that condition (7) is essential for the prediction of durability (serviceability) and is more important from the practical point of view compared to Eq. (6), which only reflects the beginning of the cracking process in concrete. In the example presented in this paper only the time to crack initiation is calculated. However, the crack width may be assessed e.g. by the model proposed by Li et al. (2006), where the key parameter is the stiffness reduction factor (Bažant and Planas 1998), taking into account concrete tensile strength, modulus of elasticity, fracture energy and crack spacing.

(ii) When the progression of corrosion and consequently the opening of cracks continue, the network of cracks is propagated, possibly reaching the surface of the concrete cover. Together with cracks due to mechanical loading, the crack network may form separating concrete elements. The concrete stress at the delaminating surface may be considered as a governing quantity. *Delamination* is a complex effect depending e.g. on the diameters of reinforcing bars, their location, concrete quality, cover, type and amount of loading, and the configuration of the structure. Such a state is either an SLS or a ULS – depending on the location and the severity of this effect. To our knowledge, an analytical model exists for the assessment of such damage derived on the basis of fracture mechanics (Li et al., 2004). However, this issue is not treated in the present paper.

(iii) A decrease in the *effective reinforcement cross-section* due to the corrosion, leading to excessive deformation, loss of bearing capacity and finally to collapse, may fall into the SLS (deformation capacity) or ULS (load bearing capacity) categories. The limit condition reads:

$$P_f(t_D) = P\left\{A\left(t_D\right) - A_{\min} \leq 0\right\} \leq P_d \qquad (8)$$

where A is the reinforcement cross-sectional area at time t_D and A_{min} is the minimum acceptable reinforcement cross-sectional area with regard to either the SLS or ULS. Both pitting and/or a uniform type of corrosion may be considered in Eq. (8).

(iv) Due to reinforcement corrosion *bond* capabilities may be significantly affected. A low level of corrosion (a diameter loss of up to about 1% – Chung et al. 2008) causes an increase in bond capacity. When the corrosion exceeds a certain level, the bond stress decreases considerably. The key parameters are, apart from corrosion level, the confinement (provided by transverse steel and concrete cover), bar diameter and applied current density (Ouglova et al. 2008, Saether 2009). It would not be practical to construct a specific limit condition in the form of Eq. (1) considering bond capacity. Instead, the gradual decrease of bond due to the actual degree of rebar corrosion (time dependant) may be reflected in changes to the bond stress-slip function and incorporated directly in the non-linear finite element code while assessing the SLS or ULS of a structure in question – a discussion of such limit states and their sensitivity to corrosion attack is provided by Zhang et al. (2009). In this respect the "bond of corroded reinforcement" is not considered as a DLS and is mentioned here only for the sake of completeness. The bond stress-slip function influenced by corrosion has been shown and some models are proposed e.g. in (Wang and Liu 2004, Maaddawy et al. 2005, Fisher et al. 2009).

The key factor for the modeling of steel corrosion as a time dependent process is the corrosion rate, which is usually expressed through current density, i_{corr}. This variable is strongly affected by ambient conditions such as humidity (Živica 1994) and temperature, moisture and oxygen availability at the level of the steel, the degree of concrete carbonation and the amount of chlorides (Morinaga 1988, Escalante and Satoshi 1990); thus depending on the diffusion characteristics of concrete (Matsushima et al. 1996) that are affected e.g. by the water to cement ratio (Vu and Stewart 2000) or cracks in the concrete cover (Schiessel and Raupach 1997). According to Alonso et al. (1988), the rate of corrosion is inversely proportional to concrete resistivity. This approach may be applied for carbonated concrete structures, but especially for chloride contaminated concrete it should be modified by several factors (Duracrete 1999) considering the influence of chloride content, galvanic effects, the formation of rust products and availability of oxygen. However, sufficient data for factor evaluation is lacking. Escalate and Satoshi (1990) have investigated the effect of oxygen concentration, chloride concentration and the pH of anodic areas on corrosion propagation. According to their studies, oxygen controls the rate of corrosion and the chloride concentration affects the amount of areas where corrosion initiates. Also, decreasing the pH of anodic areas helps to re-initiate the corrosion in the moisture cycle following the drying cycle.

Langford and Broomfield (1987) specified four ranges of resistivity for the orientation classification of corrosion rates (corresponding current densities were identified in Broomfield et al. 1993): $i_{corr} < 0.1$ μA/cm^2 is low, $0.1 < i_{corr} < 0.5$ μA/cm^2 is low to moderate, $0.5 < i_{corr} < 1$ μA/cm^2 is moderate to high and $i_{corr} > 1$ μA/cm^2 is a high rate of corrosion. When i_{corr} is not (or can not be) measured in situ then it may be treated as a random quantity to allow for the scatter expected in reality. This approach is followed in the present paper, also taking advantage of findings in (Vořechovská et al. 2009).

Software tool and numerical examples

Software tool

In the present work, modelling of degradation processes is based on simple models (often semi-empirical). The input variables are treated as random quantities; therefore, the outputs are also capable of expressing statistical and probabilistic quality with respect to time evolution. Models selected on the basis of the authors´ literature survey were primarily published as deterministic ones; for our purposes all of them were randomized, i.e. inputs were treated as random variables described by their probability density function (PDF) with proper parameters. They are included in the probabilistic software FReET (Novák et al. 2003), www.freet.cz – a combination of analytical models and simulation techniques – creating a special degradation module, FReET-D – Teplý et al. (2007) for assessing the potential degradation of newly designed as well as existing concrete structures. The implemented degradation models may serve directly in the durability assessment of structures in tasks such as the assessment of service life and the level of relevant reliability. The user may create different limit conditions. For the statistical analysis of the following examples the Latin Hypercube Sampling method was applied, although FReET also works with the crude Monte Carlo method or FORM. Numerical and/or graphic forms of outputs are available, i.e. the results of statistical analyses of degradation measures (e.g. depth of carbonation), sensitivity factors for individual inputs of the chosen model, and reliability measures (the probability of failure of the index of reliability) considering the given limit condition. For the output quantities the best fit of PDF may be found using the Kolmogorov Smirnov goodness-of-fit test (KST); also, Bayes updating is an option.

FReET-D serves for the probabilistic assessment of different DLSs encompassing together:

- 9 models for carbonation assessment in concretes from Portland or blended cements and for concrete from Portland cement with lime-cement mortar coating (outputs: carbonation depth at time *t* or time to depassivation),
- 4 models for chloride ingress modelling (variants of output: depth of chloride penetration at time *t,* time to depassivation or concentration of chlorides at depth *x* and time *t*), and
- 7 models for effects of steel corrosion (variants of output: net cross sectional area of rebar at time *t*, pit depth at time *t*, time to concrete cracking due to corrosion, width of cracks in concrete due to corrosion, and the stress intensity factor at the pit tip at time *t*).

The main criteria in selecting the degradation model for each specific use – apart from the relevant degradation mechanism and required accuracy of the model – are mostly concerned with the availability of input data (statistical data).

Some of those models or their combinations are used in the following examples to illustrate possible practical applications. The selection of input variables, their PDFs and the values of their statistical characteristics in the following illustrative examples are partially based on the authors' experience and/or on literature sources.

Carbonation depth prognosis – cooling tower

Utilizing a model based on Papadakis et al. (1992), the carbonation depth on an RC cooling tower with a height of 206 m was analyzed. The tower was investigated at the age of 19.1 years and the depth of carbonation was measured at 75 locations on both the indoor and outdoor surfaces (Keršner et al. 1996), thus providing statistical data. Table 1 provides a comparison of the analytical results with the findings of tests, namely the mean and coefficient of variation (COV). The agreement is satisfactory. For computations the input values described in Table 2 (see appendix) were used. The computational analysis also provides a prognosis for future decades which is effectively corrected by Bayess updating while utilizing the real (measured) data for the age of 19.1 years. After doing this one arrives at a lower dispersion of carbonation depth for t > 19.1 with COV = 20 % (outdoor tower surface) or 22% (indoor surface) and a more "realistic" mean value with the consequence of more reliable residual service life prediction. The comparison of the result calculated using the model and Bayess updating is plotted in Fig. 1 for the indoor surface in the range of 0 to 50 years.

Table 1 here

Figure 1 here

Crack initiation due to corrosion

In this illustrative example the calculations of both initiation and propagation periods are presented. Firstly, the time to reinforcement depassivation, t_i due to carbonation and/or chloride ingress dependent on the concrete cover thickness (this quantity is assumed here as being in the range from 25 to 60 mm) is calculated. Next, the crack initiation due to corrosion is assessed utilizing the results from depassivation calculations. The deterministic models were adopted from Papadakis et al. (1992) for carbonation, Papadakis et al. (1996) for chloride ingress and Liu and Weyers (1998) for crack initiation.

A full description of all input values for the assessment of time to depassivation is given in the appendix (Table 3). The results of the statistical analysis are shown in Fig. 2, where mean values together with standard deviations (std) are plotted. Let us focus on a concrete cover of 45 mm and apply conditions (Eqs. 4, 5) where the target design life t_D is equal to 50 years. We obtain β = 2.85 (P_f = 2×10^{-3}) and β = 0.7 (P_f = 2×10^{-1}, not an acceptable value) for depassivation due to carbonation and chloride ingress, respectively. The well known fact that the rate of chloride ingress is greater compared to the carbonation rate with respect to time to depassivation is also evident from this example.

Let us note that for concretes with supplementary cementitious materials (SCM) or with blended cements the results would be different, as the effect of partial replacement of Portland cement by SCM on the resistance to both deteriorative effects is known and reported elsewhere (Thomas and Bamforth 1999, Khunthongkeaw et al. 2006, Sisomphon and Franke 2007). Appropriate models are also encompassed in FReET-D; however, such cases are not treated in the present text due to the great variety of SCM types, their dosing and other conditions, all of which would deserve extensive study. Analyses of such a kind and a comparison of the results to experimental findings (Jiang et al. 2000, Khunthongkeaw et al. 2006) were published e.g. by Chromá et al. (2005) and Chromá et al. (2007).

Figure 2 here

In addition to the previous calculation, let us assume a time to corrosion initiation t_i due to chloride ingress for a cover of 40 mm. The best fit for the resulting t_i gained by KST is the two-parametric lognormal distribution function: t_i = LN (47.4; 11.5) years. This result can now be utilized in combination with the model for the time to crack initiation due to reinforcement corrosion, t_c. A decisive input quantity is the concrete tensile strength f_{ct}, which is considered in this study as being in the range from 3 to 10 MPa. A full description of the input parameters is given in the appendix (Table 4). The resulting time of crack initiation $t_c = t_i + t_p$, dependent on the tensile strength of concrete, is plotted in Fig. 3. Using KST it follows that for f_{ct} = 3 and 4 MPa a lognormal two-parametric PDF and for f_{ct} = 5 to 10 MPa a lognormal three-parametric PDF are the best fits of the output histograms of t_c. It is illustrated in Fig. 3.

If we apply the limit state condition given by Eq. (2) for $t_S = t_c$ and t_D = 50 years, we obtain the reliability indices plotted in Fig. 4. Assuming a design value of the reliability index of β_d = 1.3 (the recommendation of the *fib*-Model Code (2010) for a DLS) it follows from the figure that concrete with approximately f_{ct} > 7 MPa would satisfy these reliability recommendations.

Figure 3 here

Figure 4 here

Loss of reinforcement cross section due to corrosion

Let us assume that an initial reinforcement diameter is lognormally distributed (2 par): d_i = LN(30; 0.75) mm, and that the critical loss of the reinforcement area is assessed to be 10% (such a loss may e.g. lead to the exceeding of the reliability level for the ULS or SLS – depending on the structure and loading configuration). This limit corresponds to the critical net rebar diameter of 28.46 mm. The uniform type of corrosion is considered. The main input parameters of the chosen model (based on Rodrigues et al., 1996) are current density: i_{corr} = N(1; 0.2) μA/cm^2 (Vořechovská et al., 2009), and time to corrosion initiation: t_i = LN (47.4; 11.5) years, which was gained by the model for the chloride ingress used in the previous example. The decrease in rebar diameter over time is plotted in Fig. 5. The times t_i and $t_{d,crit}$ (the time of a critical drop in rebar diameter) are marked in this figure.

The best-fitted PDFs for the output net rebar diameters are LN (2 par) for 0, 10 and 20 years, Student t for 30 years, Laplace for 40 and 50 years, LN (2 par) for 60, 70, 80, 90 and 100 years, LN (3 par) for 110 and 120 years and LN (2 par) for 130, 140 and 150 years. Chosen histograms of output parameters together with fitted PDFs are depicted in Fig. 6, showing the complexity of the statistical description of the problem solved. Note that lognormal PDFs appear for time intervals of 0-20 and 60-150 years. In the first interval the steel is not yet depassivated, while in the second time interval the steel is already depassivated in the majority of stochastic realizations. Therefore, the std of the output net rebar diameter in the time interval of 0 to 20 years is influenced by the std of the input initial bar diameter only while the std in the time interval of 60-150 years is affected by the scatter of all input variables. In the time interval between these (i.e. 20-60 years) the std gradually increases (see Fig. 5).

Figure 5 here

Figure 6 here

If we apply the limit state condition given by Eq. (8), replacing the cross sections by diameters, and assume a design service life of 50 years, we obtain a reliability index of β = 0.38 (a non-acceptable level of reliability). If the reliability limit β = 1.3 is considered, the corresponding service life would be ~30 years only. The coherency of reliability level and service life is clearly demonstrated in this example.

Conclusions

- Concrete is the premier construction material and design for durability is a decisive issue in sustainability-based building strategy.
- For the effective decision making of designers and clients, the assessment of both service life and reliability level play a dominant role; such activities may be enhanced by the presented approach.
- An effective probabilistic software tool is briefly described and its utilization in numerical examples is shown, demonstrating the features of durability design of reinforced concrete structures.
- The advanced durability design approach – a probabilistic performance-based approach – is shown in the paper. Durability Limit States specialized for the initiation period and propagation period are presented, restricted to reinforcement corrosion effects, although a comprehensive choice of limit states is offered and the methodology is general enough to serve for other degradation types as well.

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Appendix

Table 2 here

Table 3 here

Table 4 here

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Table 1 Carbonation depths in a cooling tower: comparison of an analytical model with measurements on a real structure at the age of 19.1 years.

Surface	**Mean [mm]**		**COV [%]**	
	in situ measurement (Keršner et al. 1996)	FReET-D	in situ measurement (Keršner et al. 1996)	FReET-D
Outdoor (RH = 70%)	14.9	12.4	56	21
Indoor (RH = 93%)	8	11.5	29	30

Table 2 Input parameters for calculation of carbonation depths in a cooling tower

Input parameter	**Unit**	**Mean value**	**COV**	**PDF**	**Reference**
Time of exposure	years	19.1	-	Deterministic	
CO_2 content in the atmosphere	mg/m^3	800	0.12	Normal	*fib*-Model Code (2010)
Relative humidity: outdoor indoor	%	70 93	0.07 0.03	Beta (bounds a = 0, b = 100)	*fib*-Model Code (2010)
Unit content of cement in concrete	kg/m^3	342	0.03	Normal	*fib*-Model Code (2010)
Unit content of water in concrete	kg/m^3	188	0.03	Normal	*fib*-Model Code (2010)
Unit content of aggregate (0-4 mm)	kg/m^3	834	0.03	Normal	EN 206-1 (2000)
Unit content of aggregate (4-8 mm)	kg/m^3	373	0.03	Normal	EN 206-1 (2000)

Unit content of aggregate (8-16 mm)	kg/m^3	614	0.03	Normal	EN 206-1 (2000)
Specific gravity of cement in concrete	kg/m^3	3100	0.02	Normal	
Specific gravity of aggregate (0-4 mm)	kg/m^3	2590	0.02	Normal	
Specific gravity of aggregate (4-8 mm)	kg/m^3	2540	0.05	Normal	
Specific gravity of aggregate (8-16 mm)	kg/m^3	2660	0.05	Normal	
Uncertainty factor of model	-	1	0.15	Lognormal (2 par)	JCSS (2006)

Table 3 Input parameters for the calculation of time to reinforcement depassivation

Variable	**Unit**	**Mean value**	**COV**	**PDF**	**Reference**
Uncertainty factor of model	-	1	0.15	Lognormal (2 par)	JCSS (2006)
CO_2 content in the atmosphere	mg/m^3	820	0.12	Normal	*fib*-Model Code (2010)
Relative humidity	%	70	0.07	Beta (a = 0, b = 100)	*fib*-Model Code (2010)
Unit content of OPC cement	kg/m^3	313	0.03	Normal	EN 206-1 (2000)
Unit content of water	kg/m^3	185	0.03	Normal	EN 206-1

					(2000)
Unit content of aggregate (0-4 mm)	kg/m^3	847	0.03	Normal	EN 206-1 (2000)
Unit content of aggregate (4-8 mm)	kg/m^3	386	0.03	Normal	EN 206-1 (2000)
Unit content of aggregate (8-16 mm)	kg/m^3	625	0.03	Normal	EN 206-1 (2000)
Specific gravity of cement	kg/m^3	3100	0.02	Normal	
Specific gravity of aggregate (0-4 mm)	kg/m^3	2590	0.02	Normal	
Specific gravity of aggregate (4-8 mm)	kg/m^3	2540	0.02	Normal	
Specific gravity of aggregate (8-16 mm)	kg/m^3	2660	0.02	Normal	
Concrete cover	mm	25 - 60	-	Deterministic	
Concentration of Cl^- on nearest concrete surface	mol/m^3	50	-	Deterministic	Papadakis et al. (1996)
Saturation concentration of Cl^- in solid phase	mol/m^3	140	-	Deterministic	
Threshold concentration of Cl^- in liquid phase	mol/m^3	13.4	-	Deterministic	Papadakis et al. (1996)
Diffusion coefficient of Cl^- in infinite solution	m^2/s	1.6×10^{-9}	-	Deterministic	Papadakis et al. (1996)

Table 4 Input parameters for calculation of time to crack initiation due to reinforcement corrosion

Input parameter	Unit	Mean value	COV	PDF	Reference
Initial bar diameter	mm	30	-	Deterministic	
Porous zone thickness	mm	0.0125	-	Deterministic	
Concrete cover	mm	40	0.19	Lognormal (2 par)	Engelung and Faber (1999)*
Time to corrosion initiation	years	47.4	0.24	Lognormal (2 par)	
Current density	μA/cm^2	1	0.2	Normal	Vořechovská et al. (2009)
Specific gravity of rust	kg/m^3	3600	0.02	Normal	
Specific gravity of steel	kg/m^3	7850	0.01	Normal	
Ratio of steel to rust molecular weight	-	0.57	-	Deterministic	
Tensile strength of concrete	MPa	3 - 10	-	Deterministic	
Modulus of elasticity of concrete	GPa	27	0.08	Lognormal (2 par)	
Poisson´s ratio of concrete	-	0.18	-	Deterministic	
Creep coefficient	-	2	-	Deterministic	
Uncertainty factor of model	-	1	0.15	Lognormal (2 par)	JCSS (2006)

*only type of PDF, not COV

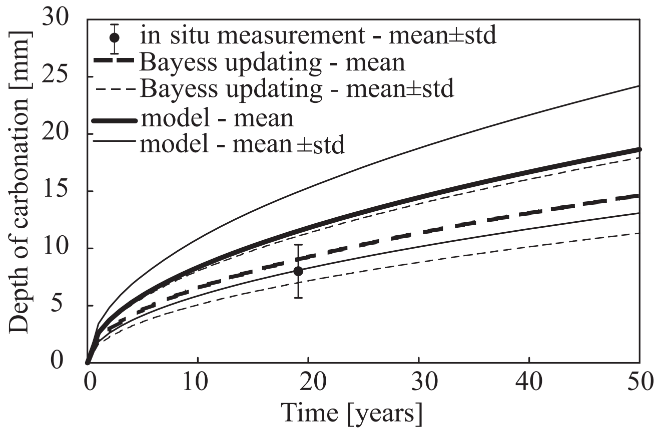

Figure 1 The comparison of carbonation model results and Bayess updating for the indoor surface.

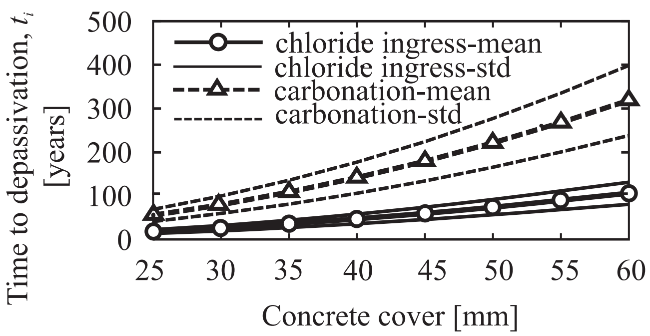

Figure 2 Time to depassivation (mean ± std) due to carbonation and chloride ingress vs. concrete cover.

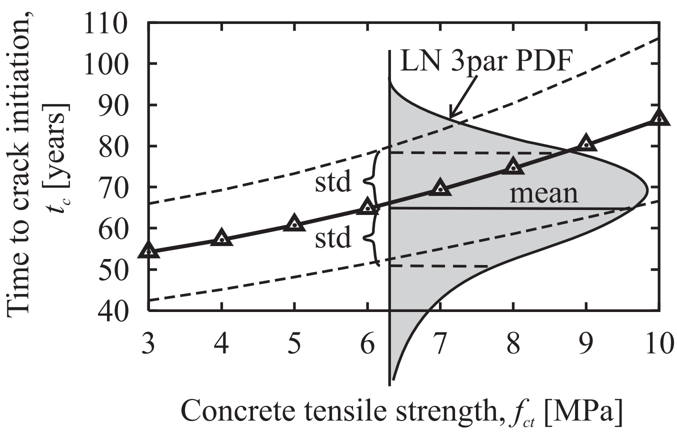

Figure 3 Time to crack initiation (± std) vs. tensile strength of concrete.

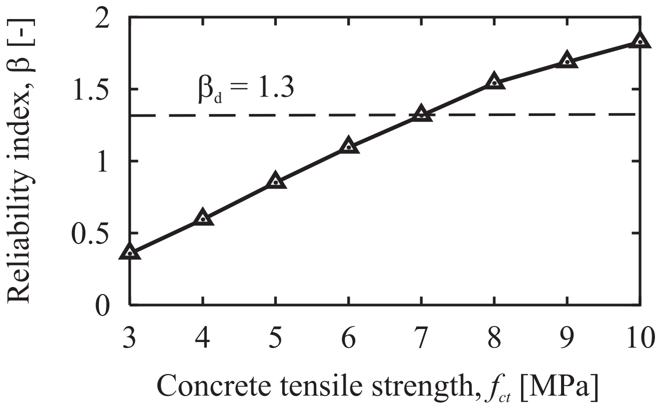

Figure 4 Reliability indices for t_D = 50 years vs. tensile strength of concrete.

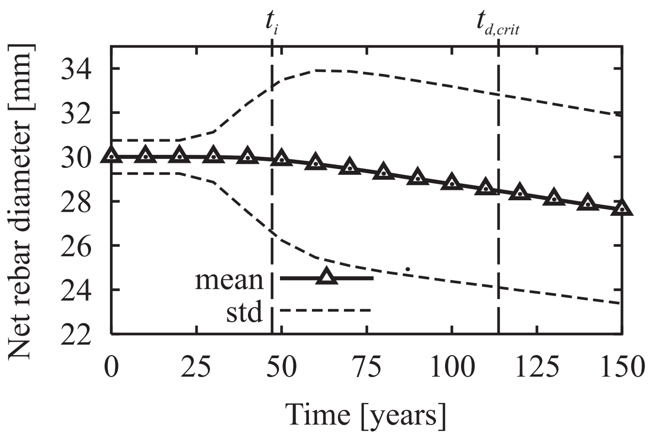

Figure 5 Net rebar diameter (mean ± std) vs. time.

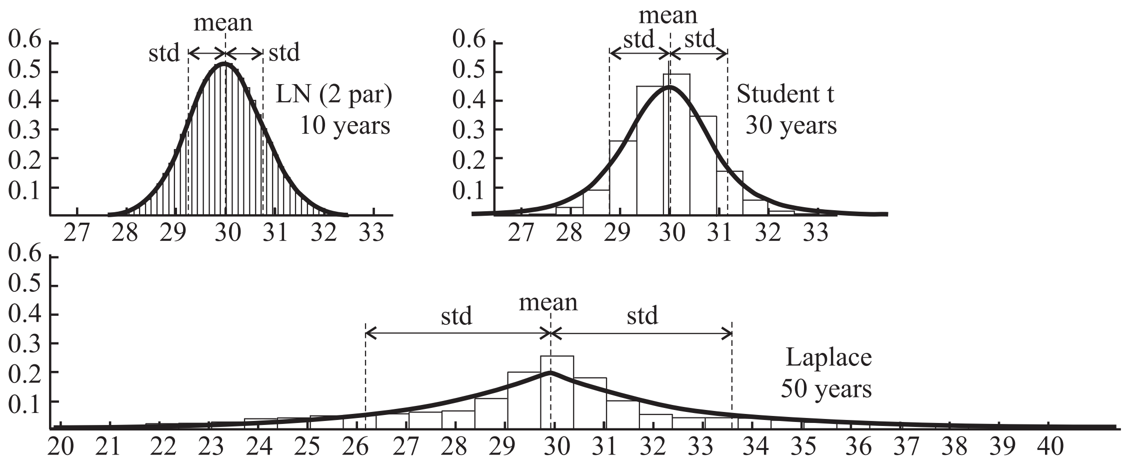

Figure 6 Histograms of output net rebar diameter plotted in Fig. 5 (decreased due to corrosion) together with the best-fitted PDFs in chosen time steps.